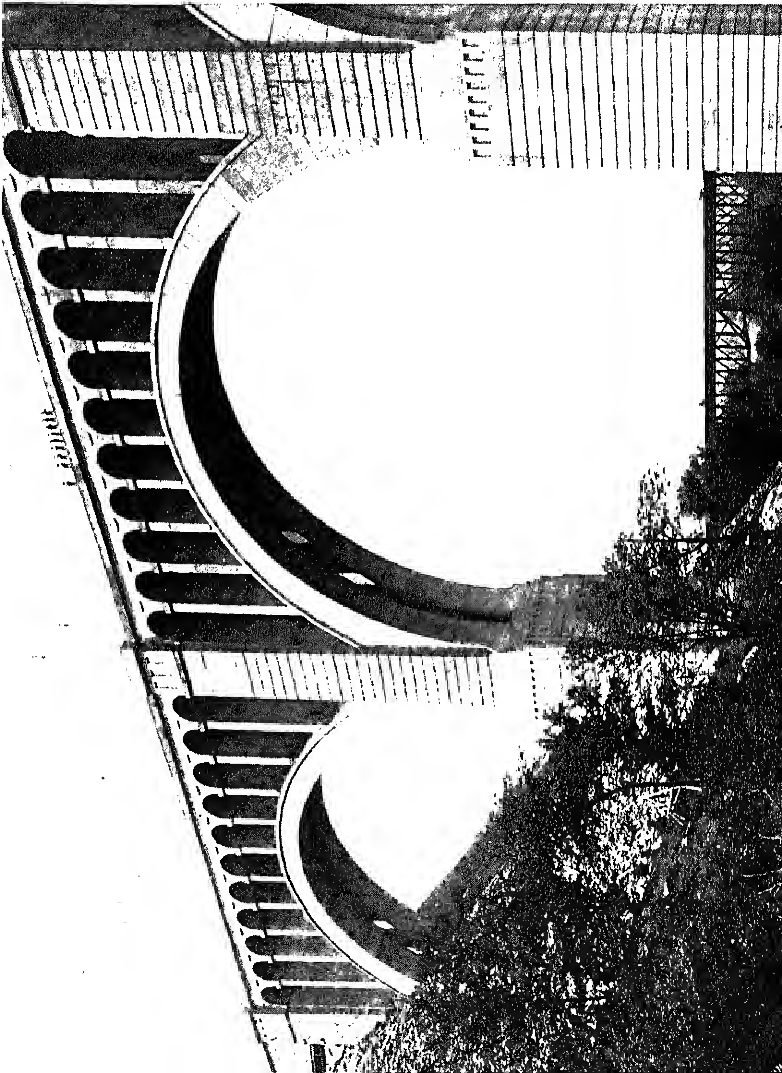

Cyclopedia
of
Civil Engineering

A General Reference Work on

SURVEYING, HIGHWAY CONSTRUCTION, RAILROAD ENGINEERING, EARTHWORK, STEEL CONSTRUCTION, SPECIFICATIONS, CONTRACTS, BRIDGE ENGINEERING, MASONRY AND REINFORCED CONCRETE, MUNICIPAL ENGINEERING, HYDRAULIC ENGINEERING, RIVER AND HARBOR IMPROVEMENT, IRRIGATION ENGINEERING, COST ANALYSIS, ETC.

Prepared by a Corps of

CIVIL AND CONSULTING ENGINEERS AND TECHNICAL EXPERTS OF THE HIGHEST PROFESSIONAL STANDING

Illustrated with over Two Thousand Engravings

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sire to express their indebtedness, particularly, to the
eminent authorities, whose well-known treatises should be in the
everyone interested in Civil Engineering.

Grateful acknowledgment is here made also for the inval
operation of the foremost Civil, Structural, Railroad, Hydraulic, an
Engineers and Manufacturers in making these volumes thoroug
sentative of the very best and latest practice in every branch of
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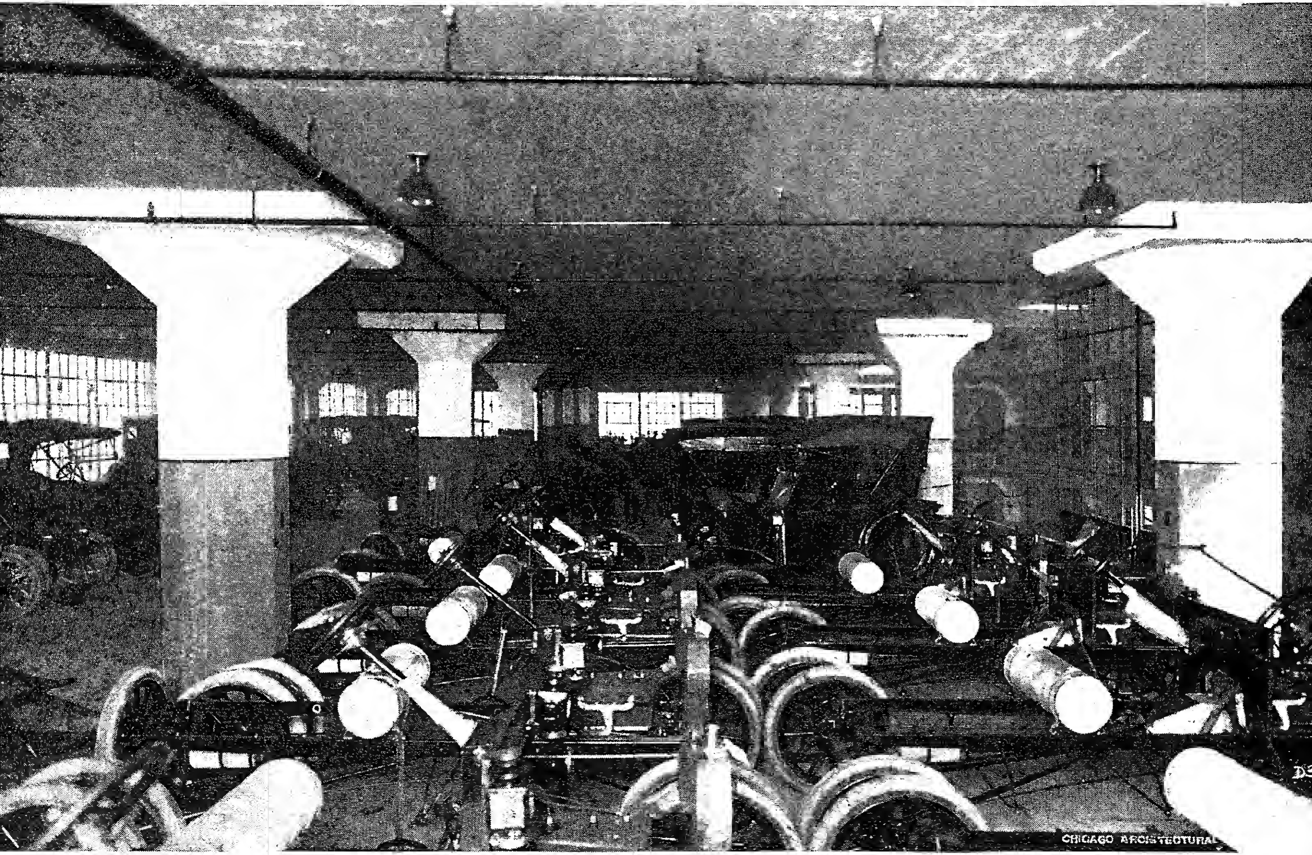


INTERIOR OF CHICAGO SERVICE BUILDING OF FORD MOTOR COMPANY

Foreword

OF all the works of man in the various branches of engineering, none are so wonderful, so majestic, so awe-inspiring as the works of the Civil Engineer. It is the Civil Engineer who throws a great bridge across the yawning chasm which seemingly forms an impassable obstacle to further progress. He designs and builds the skeletons of steel to dizzy heights, for the architect to cover and adorn. He burrows through a great mountain and reaches the other side within a fraction of an inch of the spot located by the original survey. He scales mountain peaks, or traverses dry river beds, surveying and plotting hitherto unknown, or at least unsurveyed, regions. He builds our Panama Canals, our Arrow Rock and Roosevelt Dams, our water-works, filtration plants, and practically all of our great public works.

¶ The importance of all of these immense engineering projects and the need for a clear, non-technical presentation of the theoretical and practical developments of the broad field of Civil Engineering has led the publishers to compile this great reference work. It has been their aim to fulfill the demands of the trained engineer for authoritative material which will solve the problems in his own and allied lines in Civil Engineering, as well as to satisfy the desires of the self-taught practical man who attempts to keep up with modern engineering developments.

¶ Books on the several divisions of Civil Engineering are many and valuable, but their information is too voluminous to be of the greatest value for ready reference. The Cyclopedia of Civil Engineering offers more condensed and less technical treatments of these same subjects from which all unnecessary duplication has been eliminated; when compiled into nine handy volumes, with comprehensive indexes to facilitate the looking up of various topics, they represent a library admirably adapted to the requirements of either the technical or the practical reader.

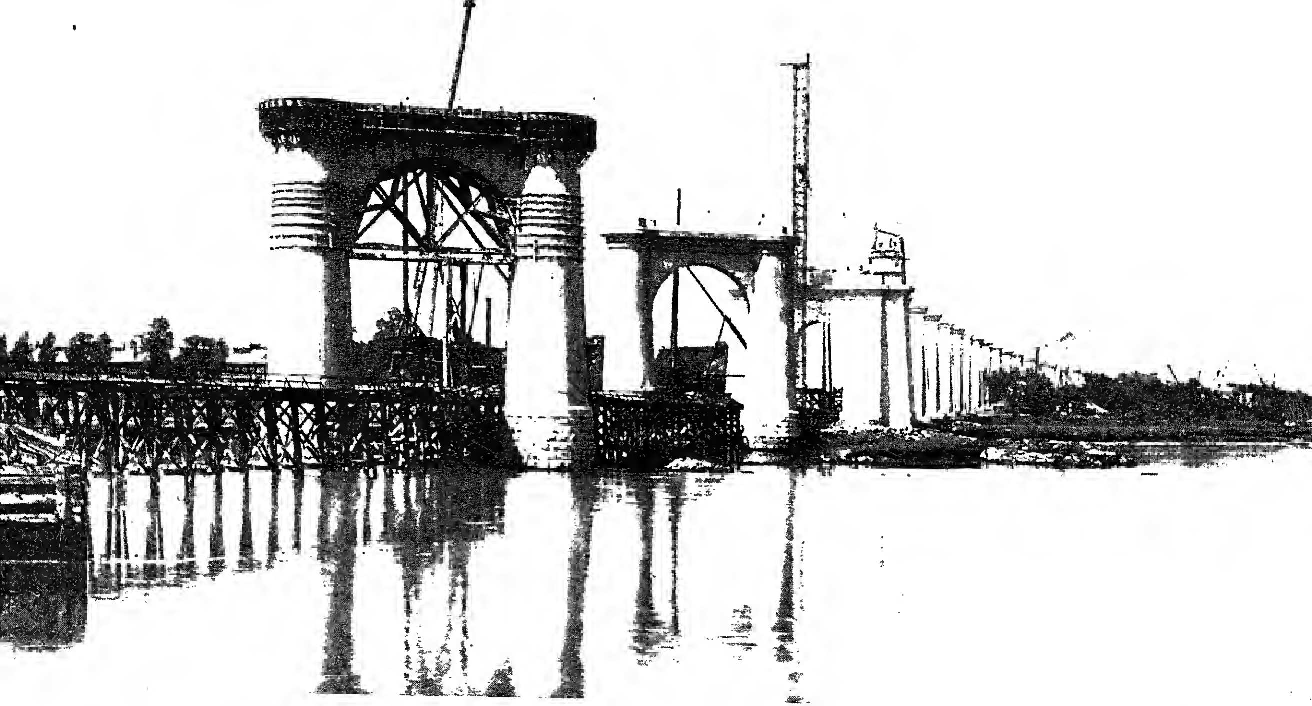
¶ The Cyclopedia of Civil Engineering has for years occupied an enviable place in the field of technical literature as a standard reference work and the publishers have spared no expense to make this latest edition even more comprehensive and instructive.

¶ In conclusion, grateful acknowledgment is due to the staff of authors and collaborators—engineers of wide practical experience, and teachers of well recognized ability — without whose hearty co-operation this work would have been impossible.

VOLUME VI

* For page numbers, see foot of pages.

† For professional standing of authors, see list of Authors and Collaborators at front of volume.



VIEW OF REINFORCED CONCRETE ABUTMENTS FOR THE NEW HELL GATE BRIDGE OVER EAST RIVER, NEW YORK

The bridge and approaches will be 3½ miles long, connecting the New York, New Haven and Hartford system with the Pennsylvania system, thus avoiding a long ferry connection. The bridge consists of seven sections: (1) A plate girder span structure, total length 4,356 feet; (2) Bronx kill bridge section, two spans of basculc type 490 feet long; (3) Randall's Island viaduct plate girder spans 1,965 feet long; (4) Large Hell Gate bridge, spanning East River, 1,154 feet long; (5) Ward's Island viaduct 2,054 feet long; (6) Hell Gate bridge, 1,150 feet long, greatest steel arch span in the world; (7) Long Island viaduct, plate girders and concrete piers 6,352 feet long.

MASONRY AND REINFORCED CONCRETE

PART I

MASONRY MATERIALS

Masonry may be defined as construction in which the chief constructive material is stone or an artificial mineral product such as brick, terra cotta, or cemented blocks. Under this broad definition, even reinforced concrete may be considered as a specialized form of masonry construction.

NATURAL STONE

BUILDING VARIETIES

Limestone. Carbonate of lime forms the principal ingredient of limestone. A pure limestone should consist only of carbonate of lime. However, none of our natural stones are chemically pure, but all contain a greater or less amount of foreign material. To these impurities are due the beautiful and variegated coloring which makes limestone valuable as a building material.

Limestone occurs in stratified beds, and ordinarily is regarded as originating as a chemical deposit. It effervesces freely when an acid is applied; its texture is destroyed by fire; the fire drives off its carbonic acid and water, and forms quicklime. Limestone varies greatly in its physical properties. Some limestones are very durable, hard, and strong, while others are very soft and easily broken.

There are two principal classes of limestone—*granular* and *compact*. In each of these classes are found both marble and ordinary building stone. The granular stone is generally best for building purposes, and the finer-grained stones are usually better for either marble or fine cut-stone. The coarse-grained varieties often dis-

for ornamental purposes, it is called *marble;* or, in other words, any limestone that can be polished is called marble. There are a great many varieties of marble, and they vary greatly in color and appearance. Owing to the cost of polishing marble, it is used chiefly for ornamental purposes.

Dolomite. When the carbonate of magnesia occurring in limestone rises to about 45 per cent, the stone is then called *dolomite.* It is usually whitish or yellowish in color, and is a crystalline granular aggregate. It is harder than the ordinary limestones, and also less soluble, being scarcely at all acted upon by dilute hydrochloric acid. There is no essential difference between limestone and dolomite with respect to color and texture.

Sandstone. Sandstones are composed of grains of sand that have been cemented together through the aid of heat and pressure, forming a solid rock. The cementing material usually is either silica, carbonate of lime, or an iron oxide. Upon the character of this cementing material is dependent, to a considerable extent, the color of the rock and its adaptability to architectural purposes. If silica alone is present, the rock is of a light color and frequently so hard that it can be worked only with great difficulty. Such stones are among the most durable of all rock, but their light color and poor working qualities are a drawback to their extensive use. Rock in which carbonate of lime is the cementing material is frequently too soft, crumbling and disintegrating rapidly when exposed to the weather. For many reasons the rocks containing ferruginous cement (iron oxide) are preferable. They are neither too hard to work readily nor liable to unfavorable alteration, when exposed to atmospheric agencies. These rocks usually have a brown or reddish color.

Sandstones are of a great variety of colors, which, as has already been stated, is largely due to the iron contained in them. In texture, sandstones vary widely—from a stone of very fine grain, to one in which the individual grains are the size of a pea. Nearly all sandstones are more or less porous, and hence permeable to a certain extent by water and moisture. Sandstones absorb water most readily in the direction of their lamination or grain. The strength and hardness of sandstones vary between wide limits. Most of the varieties are easily worked, and split evenly. The formations of

sandstone in the United States are very extensive. The crushing strength of sandstone varies widely, being from 2500 pounds to 13,500 pounds per square inch, and specimens have been obtained that require a load of 29,270 pounds per square inch to crush them.

Conglomerates. Conglomerates differ from sandstone only in structure, being coarser and of a more uneven texture. The grains are usually an inch or more in diameter.

Granite. The essential components of the true granites are quartz and potash feldspar. Although the essential minerals are but two in number, granites are rendered complex by the presence of numerous accessories which essentially modify the appearance of the rocks; and these properties render them important for building stone. The prevailing color is some shade of gray, though greenish, yellowish, pink, and deep red are not uncommon. These various hues are due to the color of the prevailing feldspar and the amount and kind of the accessory minerals. The hardness of granite is due largely to the condition of the feldspathic constituent, which is valuable. Granites of the same constituents differ in hardness.

Granites do not effervesce with acids, but emit sparks when struck with steel. They possess the properties of strength, hardness, and durabilty, although they vary in these properties as well as in their structure. They furnish an extensive variety of the best stone for the various purposes of the engineer and the architect. The crushing strength of granite is variable, but usually is between 15,000 and 20,000 pounds per square inch.

Trap Rock. Trap rock, or *diabase*, is a crystalline, granular rock, composed essentially of feldspar and augite; but nearly all contains magnetite and frequently olivine. It is basic in composition and in structure; as a rule, it is massive. The texture, as a general thing, is fine, compact, and homogeneous. The colors are somber, varying from greenish, through dark gray, to nearly black. Owing to its lack of rift, its hardness, and its compact texture, trap rock is generally very hard to work. It has been used to some extent for building and monumental work, but is more generally used

From the constructor's standpoint, any [illegible] will fulfil certain desired characteristics. The [illegible] teristics are not found combined in the highest [illegible] kind of stone. It is essential to learn to what [illegible] desirable characteristics are combined in the [illegible] which are quarried. At the same time, it [illegible] that stones of the same nominal classification [illegible] extent of their desirability. The chief characteristics [illegible] sidered by the constructor are *cost, durability,* [illegible] *ance.* Although in some cases this represents [illegible] these qualifications are desired, in other cases the [illegible] varied. For example, in a high-grade public building [illegible] a good appearance is considered essential, regardless [illegible] subsurface foundation, appearance is of absolutely [illegible]

Cost. The cost of any stone depends on its [illegible] tion in the quarry, the cost of quarrying and dressing [illegible] of transportation from the quarry to the site of the [illegible] cost of transportation is often the most important [illegible] sideration frequently decides not only the choice [illegible] the type of construction whether stone or [illegible]

To give some idea of the cost of stone quarrying [illegible] mate costs will be given. The cost of quarrying [illegible] sandstone for heavy retaining walls will be from [illegible] ton loaded on cars; but if this same stone is wanted [illegible] work, the cost of quarrying will be increased [illegible] regular shaped stones being required, which will [illegible] waste. The cost of getting out and loading [illegible] 90 cents to \$3.00 or \$4.00 per ton, depending on the [illegible] quarry and the size and shape of the stone required.

Stone that can be quarried by the use of wedges [illegible] powder can be marketed at a small cost, but when [illegible] means are required, the cost of the stone will be [illegible] portation is generally an item in the cost of [illegible] considered and often it proves to be a serious one.

Durability. Under many conditions the most [illegible] fication is durability. The lack of it is also the [illegible] disappointing quality. Rocks which [illegible]

disintegrate after a comparatively few years' exposure.

Atmospheric Influences. A very porous stone will absorb water, which may freeze and cause crystals near the surface to flake off. Even though such action during a single winter may be hardly perceptible, the continued exposure of fresh surfaces to such action may sooner or later cause a serious loss and disintegration. Even rain water which has absorbed carbonic acid from the atmosphere will soak into the stone, and the acid will have a greater or less effect on nearly all stones. Quartz is the only constituent which is absolutely unaffected by acid. The sulphuric acid gas given off by coal will also affect building stone very seriously.

Fire. Natural stone is far less able to withstand a conflagration than the artificial compositions such as brick, concrete, and terra cotta. Granite, so popularly considered the type of durability, is especially affected. Limestone and marble will be utterly spoiled, at least in appearance, if not structurally, by a hot fire. Sandstone is the least affected of the natural stones.

Hardness. The durability of a stone is tested by its resistance to abrasive action in pavements, doorsills, and similar cases. The value of trap rock for macadam and block pavements is chiefly due to this quality.

Strength. In some structural work (as, for example, an arch) the crushing strength of the stone is the primary consideration. The average crushing strength of various kinds of stone will be quoted later. The tensile strength should never be depended on, except to a very limited extent, as a function of the transverse strength. Even this is only applicable to such cases as the lintels over doors and windows, the footing stones for foundations, and the cover stones for box culverts. It is usually true that a stone which is free from cracks and which has a high crushing strength also, has as much transverse strength as should be required of any stone.

Appearance. It is seldom that an engineer need concern himself with the appearance of a stone, provided it is satisfactory in the respects previously mentioned. The presence of iron oxide in a stone will sometimes cause a deterioration in appearance by the formation of a reddish stain on the outer surface. It usually happens, however, that a stone whose strength and durability are satisfactory

Physical Properties of Some Building Stones*

Kind of Stone	Locality	Position	Strength per Square Inch (lbs.)	Specific Gravity	Weight per Cubic Foot (lbs.)	Ratio of Absorption
Granite	Grape Creek, Colo.	Bed Edge	14,492 17,352	2.603	163	.048
Granite	Stony Creek, Conn.	Bed Edge	15,000 16,750	2.645	165	$\frac{1}{201}$
Granite	Milford, Conn.	.....	22,610	.....	.....	....
Granite	City Point, Me.	Bed	15,046	2.65	160	....
Granite	East St. Cloud, Minn.	Bed Edge	28,000 26,250	2.609	163	....
Diabase	New Duluth, Minn.	Bed Edge	26,250 26,250	3.005	188	$\frac{1}{338}$
Limestone	Bedford, Ind.	.....	6,500	.....	147	$\frac{1}{24}$
Limestone	Bedford, Ind.	.....	10,125	.....	152	$\frac{1}{32}$
Limestone	Greensburgh, Ind.	.....	16,875	.....	170	$\frac{1}{117}$
Limestone	Conshohocken, Pa.	.....	15,150	.....	.....	....
Limestone	Stillwater, Minn.	.....	25,000	2.762	173	$\frac{1}{251}$
Limestone	Stillwater, Minn.	Bed Edge	10,750 12,750	2.567	161	$\frac{1}{40}$
Sandstone	Buckhorn, Larimer Co., Colo.	Bed Edge	18,573 17,261	2.379	168	.040
Sandstone	Fort Collins, Larimer Co., Colo.	Bed Edge	11,707 10,784	2.252	141	.072
Sandstone	Brandford, Fremont Co., Colo.	Bed Edge	3,308 2,894	2.004	125	....
Sandstone	Marquette, Mich.	Bed	6,323	2.166	135	$\frac{1}{20}$
Sandstone	Kasota, Minn.	Bed	10,700	2.630	164	$\frac{1}{56}$
Sandstone	Albion, N. Y.	Bed	13,500	2.420	151	$\frac{1}{44}$
Sandstone	Cleveland, O.	Bed	6,800	2.240	140	$\frac{1}{37}$
Sandstone	Seneca, O.	Bed	9,687	2.390	149	$\frac{1}{32}$

*From Merrill's "Stone for Buildings and Decoration".

tectural work, where it is considered essential that a certain color or appearance shall be obtained.

Seasoning of Stone. Stone, to weather well, should be laid with its bedding laminations horizontal, as it was first laid down by Nature in the quarry. The stone, moreover, will offer greater resistance to pressure if laid in this manner, and, it is said, will stand a greater amount of heat without disintegrating. This is important in cities where any building is liable to have its walls highly heated by neighboring burning structures.

Some stones that are liable to be destroyed by the effects of frost on first being taken from the quarries, are no longer so after being exposed for some time to the air, having lost their quarry water through evaporation. This difference is very manifest between stones quarried in summer and those quarried in winter. It has frequently happened that stones of good quality have been entirely ruined by hard freezing immediately after being taken from the quarry; while, if they are quarried during the warm season of the year and have an opportunity to lose their quarry water by evaporation prior to cold weather, they withstand freezing very well. This particularly applies to some marbles and limestones. This change is accounted for by the claim put forward that the quarry water of the stones carries in solution carbonate of lime and silica, which is deposited in the cavities of the rock as evaporation proceeds. Thus additional cementing material is added, rendering the rock more compact. This also will account for the hardening of some stones after being quarried a short time. When first quarried they are soft, and easily sawed and worked into any desirable shape; but after the evaporation of their quarry water, they become hard and very durable.

Table I gives the physical properties of many of the most important varieties and grades of building stone found in the United States.

TESTS

Of the above four qualities, only two—durability and strength—are susceptible of laboratory tests, and even for these qualities the best known laboratory tests are not conclusive. The deterioration and partial failure of the masonry in some of the best known

in their construction, are startling illustrations of the impracticability of determining from laboratory tests the effect on stone of long-continued stress, combined perhaps with other destructive influences. Although the best technical advice was obtained in selecting the stone for the Parliament House in London, and the stone selected was undoubtedly subjected to the best known tests, it was apparently impossible to foresee the effect of the London atmosphere, which is now so seriously affecting the stone. Several of the tests to be described below should be considered as *negative tests*. If the stones fail under these tests, they are probably inferior; if they do not fail, they are perhaps safe, but there is no certainty. A long experience, based on a knowledge of the characteristics of stones which have proven successful, is of far greater value than a dependence on the results of laboratory tests. The tests attempt to stimulate the actual destructive agencies as far as possible, but since a great deal of stonework, which was apparently satisfactory when constructed and for a few years after, has failed for a variety of reasons, attempts are made to use *accelerated tests*, which are supposed by their concentration to affect the stone in a few minutes or hours as much as the milder causes acting through a long period of years.

Absorption. It is generally said that stones having the least absorption are the best. The absorptive power is measured by first drying the stone for many hours in an oven, weighing it, then soaking it for, say, 24 hours, and again weighing it. The *increase* in the weight of the soaked stone (due to the weight of water absorbed), divided by the weight of the dry stone, equals the *ratio of absorption*. The granites will absorb as an average value a weight of water equal to about $\frac{1}{150}$ of the weight of the stone. For sandstone the ratio is about $\frac{1}{24}$.

The test for absorption has but little value except to indicate a closeness of grain (or the lack of it), which *probably* indicates something about the strength of the stone, as well as its liability to some kinds of disintegration.

Test for Frost. The only real test is to wash, dry, and weigh test specimens very carefully; then soak them in water and expose them to intensely cold and intensely warm temperatures alternately. Finally wash, dry, and weigh them. If the freezing has resulted in

in weight or the breakage will give a measure of the effect of cold winters. However, as such low temperatures cannot be produced artificially except at considerable expense, and as a sufficient degree of natural cold is ordinarily unobtainable when desired, such a test is usually impracticable.

An attempt to simulate such an effect by boiling the specimen in a concentrated solution of sulphate of soda and observing the subsequent disintegration of the stone, if any, is known as *Brard's test.* Although this method is much used for lack of a better, its value is doubtful and perhaps deceptive, since the effect is largely chemical rather than mechanical. The destructive effect on the stone is usually greater than that of freezing, and might result in condemning a really good stone.

Chemical Test. The most difficult and uncertain matter to determine is the probable effect of the acids in the atmosphere. These acids, dissolved in rain water, soak into the stone and combine with any earthy matter in the stone, which then leaches out, leaving small cavities. This not only results in a partial disintegration of the stone, but also facilitates destruction by freezing. If the stone specimen, after being carefully washed, is soaked for several days in a one per cent solution of sulphuric and hydrochloric acid, the liquid being frequently shaken, the water will become somewhat muddy, if there is an appreciable amount of earthy matter in the stone. Such an effect is supposed to indicate the probable action of a vitiated atmosphere. Of course it should be remembered that such a consideration is important only for a structure in a crowded city where the atmosphere is vitiated by poisonous gases discharged from factories and from all chimneys.

Physical Tests. A test made by crushing a block of stone in a testing machine is apparently a very simple and conclusive test, but in reality the results are apt to be inconclusive and even deceptive. This is due to the following reasons, among others:

(*a*) The crushing strength of a cube per square inch is far less than that of a slab having considerably greater length and width than height.

(*b*) The result of a test depends very largely on the preparation of the specimen. If sawed, the strength will be greater than if cut by chipping. If the upper and lower faces are not truly parallel, so that there is a concentration of pressure on one corner, the apparent result will be less.

(c) The result depends on the ... and ground with machines that will insure truly parallel and plane surfaces will give higher results than when wood, lead, leather, or plaster-of-Paris cushions are employed.

(d) The strength of masonry depends largely on the crushing strength of the mortar used and the thickness of the joints. Other things being equal, an increase in the crushing strength of the stone (or brick) which is used does not add proportionately to the strength of the masonry as a whole, and if the mortar joints are very thick, it adds little or nothing. Since the strength of the masonry is the only real criterion, the strength of a cube of the stone is of comparatively little importance.

In short, tests of two-inch cubes (the size usually employed) are valuable chiefly in comparing the strength of two or more different kinds of stones, all of which are tested under precisely similar conditions. A comparison of such figures with the figures obtained by others will have but little value unless the precise conditions of the other tests are accurately known. Under any conditions, the results of the tests will bear but little relation to the actual strength of the masonry to be built.

Quarry Examinations. These are generally the surest test, and they should never be neglected, if the choice of stone is a matter of great importance. *Field stone* and outcropping rock, which have withstood the weather for indefinite periods of years, can usually be relied on as being durable against all deterioration except that due to acids in the atmosphere, to which they probably have not been subjected in the country as they might be in a city. On the other hand, however, large blocks of stone can seldom be obtained from field stones. If a quarry has been opened for several years, a comparison of the other surfaces with those just exposed may indicate the possible disintegrating or discoloring effects of the atmosphere. A stone which is dense and of uniform structure, and which will not disintegrate, may be relied on to withstand any physical stress to which masonry should be subjected.

BRICK

Definition and Characteristics. The term *brick* is usually applied to the product resulting from burning molded prisms of clay in a kiln, at a high temperature.

Common brick is not extensively used in engineering structures except in the construction of sewers and the lining of tunnels. Bric

is easily worked into structures of any desirable shape, easily handled or transported, and comparatively cheap. When well constructed, brick masonry compares very well in strength with stone masonry, but is not as heavy as stone. Brickwork is but slightly affected by changes of temperature or humidity.

Brick is made of common clay (silicate of alumina), which usually contains compounds of lime, magnesia, and iron. Good brick clay is often found in a natural state. The quality of the brick depends greatly on the quality of the clay used, and equally as much on the care taken in its manufacture.

Oxide of iron gives brick hardness and strength. The red color of brick is also due to the presence of iron. The presence of carbonate of lime in the clay of which brick is made is injurious, since the carbonate is decomposed during the burning, forming caustic potash, which, by the absorption of water, will cause the brick to disintegrate. An excess of silicate of lime makes the clay fusible, which softens the brick and thereby causes distortion during the burning process. Magnesia in small quantities has but little influence on brick. Sand, in quantities not in excess of about 25 per cent, will help to preserve the form of the brick, and is beneficial to that extent; but in greater quantities than 25 per cent, it makes the brick brittle and weak.

Requisites for Good Brick. Good brick should be of regular shape, with plane faces, parallel surfaces, and sharp edges and angles. It should show a fine, uniform, compact texture; should be hard and, when struck a sharp blow, should ring clearly; and should not absorb more water than one-tenth of its weight. The specific gravity should be 2 or more. Good brick will bear a compressive load of 6000 pounds per square inch when the sides are ground flat and pressed between plates. The modulus of rupture under transverse stress should be at least 800 pounds per square inch.

Absorptive Power. The amount of water that brick absorbs is very important in indicating the durability of brick, particularly its resistance to frost. Very soft brick will absorb 25 to 30 per cent of their weight of water. Weak, light-red ones will absorb 20 to 25 per cent; this grade of brick is used commonly for filling interior walls. The best brick will absorb only 4 to 5 per cent, but brick

molding sand, and the amount of air admitted to the kiln also have their influence. Pure clay, or clay mixed with chalk, will produce white brick. Iron oxide and pure clay will produce a bright red brick when burned at a moderate heat. Magnesia will produce brown brick; and when it is mixed with iron, produces yellow brick. Lime and iron in small quantities produce a cream color; an increase of lime produces brown, and an increase of iron, red.

Size and Weight. The standard size for common brick is $8\frac{1}{4}$ by 4 by $2\frac{1}{4}$ inches; and for face brick, $8\frac{3}{8}$ by $4\frac{1}{8}$ by $2\frac{1}{4}$ inches. There are numerous small variations from these figures; and also, since the shrinkage during burning is very considerable and not closely controlled, there is always some uncertainty and variation in the dimensions. Bricks will weigh from 100 to 150 pounds per cubic foot according to their density and hardness, the harder bricks being, of course, the heavier per unit of volume.

Classification of Common Brick. Brick are usually classified in three ways: (*a*) manner of molding; (*b*) position in kiln; (*c*) their shape or use.

(*a*) The manner in which brick is molded has produced the following terms:

Soft-Mud Brick. A brick molded either by hand or by machine, in which the clay is reduced to mud by adding water.

Stiff-Mud Brick. A brick molded from dry or semi-dry clay. It is molded by machinery.

Pressed Brick. A brick molded by machinery with semi-dry or dry clay.

Re-Pressed Brick. A brick made of soft mud, which, after being partly dried, is subjected to great pressure.

(*b*) The classification of brick with regard to their position in the kiln applies only to the old method of burning. With the new methods, the quality is nearly uniform throughout the kiln. The three grades taken from the old-style kiln were:

Arch Brick. Brick forming the sides and top of the arches in which the fire is built are called arch brick. They are hard, brittle, and weak from being overburnt.

Body, Cherry, or Hard Brick. Brick from the interior are called body, cherry, or hard brick, and are of the best quality.

Pale, Salmon, or Soft Brick. Brick forming the exterior of the kiln are underburnt, and are called soft, salmon, or pale brick. They are used only for filling, being too weak for ordinary use.

(c) The classification of brick in regard to their use or shape has given rise to the following terms:

Face Brick. Brick that are uniform in size and color and are suitable for the exposed places of buildings.

Sewer Brick. Common hard brick, smooth and regular in form.

Paving Brick. Very hard common vitrified brick, often made of shale. They are larger than the ordinary brick, and are often called *paving blocks*.

Compass Brick. Brick having four short edges which run radially to an axis. They are used to build circular chimneys.

Voussoir Brick. Brick having four long edges running radially to an axis. They are used in building arches.

Crushing Strength. The results of crushing tests of brick vary greatly, depending on the details of the tests made. Many reports fail to give the details under which these tests are made, and in that case the real value of the results of the test as announced is greatly reduced.

The following results were obtained at the U. S. Arsenal at Watertown, Mass., by F. E. Kidder. The specimens were rubbed on a revolving bed until the top and bottom faces were perfectly true and parallel.

Make of Brick	No. of Specimens Tested	Pressure at Which Specimens Began to Fail	Ultimate Compression (per sq. in.)
Philadelphia Face Brick	3	3,527 lbs.	5,918 lbs.
Cambridge Brick	4	4,655 "	12,186 "
Boston Brick	3	7,880 "	11,670 "
New England Pressed	4	4,764 "	12,490 "

The following results were obtained by C. Y. Davis, the tests being made at the Watertown Arsenal:

Kind of Brick	Compression (per sq. in.)	Kind of Brick	Compression (per sq. in.)	Kind of Brick	Compression (per sq. in.)
Red	9,540 lbs.	Pressed	6,470 lbs.	Arch	7,600 lbs.
"	*8,530 "	"	*9,190 "	"	*10,290 "
"	6,050 "	"	5,960 "	"	6,800 "
"	6,700 "	"	6,750 "		

These specimens were tested to select brick for the U. S. Pension Office at Washington, D. C. The specimens tested were submitted by manufacturers.

Fire Brick. Furnaces must be lined with a material which is even more refractory than ordinary brick. The oxide and sulphide of iron, which are so common (and comparatively harmless) in

*Indicates the brick selected.

extent than a very few per cent. Fire brick ... nearly pure sand and clay. There is ... mechanical strength, but the chief requirement ... and pure clay and silica fulfil this requirement ...

Sand-Lime Brick. Within the last few years ... brick industry has been developed to some extent. ... for making this brick consist of sand and lime ... made by molding ordinary lime mortar in the ... and were hardened by the carbon dioxide of the ...

There are two general methods of manufacture ...

(*a*) Brick made of sand and lime, and hardened ... hardening may be hastened by placing the brick ... dioxide; or still less time will be required ... dioxide under pressure.

(*b*) Brick made of sand and lime, and hardened ... pheric pressure. This process may be hastened ... pressure.

When sand-lime bricks are made by the first ... several weeks for the bricks to harden, ... it requires only a few hours; the latter method ... used in this country. The advantages claimed ... that they improve with age; are more uniform ... color; have a low porosity and no efflorescence, ... grate by freezing. The compressive strength of ... a good quality ranges from 2500 to 4000 pounds ...

CONCRETE BUILDING BLOCKS

Types. The growth of the concrete block ... rapid. The blocks are taking the place of ... for ordinary wall construction. They are ... The blocks are made at a factory or on the ... are to be used, and are placed in the wall in ... or stone. There are two general types of block, ... *block*, and the *two-piece block*. The one-piece ... block, with hollow cores, making the whole ... the two-piece type, the front and back of the ... separate pieces and bonded when laid up ... piece blocks are more generally used than ...

Size. Various shapes and sizes of blocks are made. Builders of some of the standard machines have adopted a standard length of 32 inches and a height of 9 inches for the full-sized blocks, with width of 8, 10, and 12 inches. Lengths of 8, 12, 16, 20, and 24 inches are made from the same machine, by the use of parting plates and suitably divided face-plates. Most machines are constructed so that any length between 4 and 32 inches, and any desired height, can be obtained.

The size of the openings (the cores) varies from one-third to one-half of the surface of the top or bottom of the block. The building laws of many cities state that the openings shall amount to only one-third of the surface. For any ordinary purpose, blocks with 50 per cent open space are stronger than necessary.

Materials. The material for making concrete blocks consists of Portland cement, sand, and crushed stone or gravel. Owing to the narrow space to be filled with concrete, the stone and gravel are limited to one-half or three-quarters of an inch in size. At least one-third of the material, by weight, should be coarser than $\frac{1}{8}$ inch. A block made with gravel or screenings (sand to $\frac{3}{4}$-inch stone), with proportions of 1 part Portland cement to 5 parts screenings, will be as good as a block with 1 part Portland cement and 3 parts sand. These materials will be further treated under the headings of "Portland Cement", "Sand", and "Stone".

Proportions. The proportions generally used in the making of concrete blocks vary from a mixture of 1 part cement, 2 parts sand, and 4 parts stone, to a mixture of 1 part cement, 3 parts sand, and 6 parts stone. A very common mixture consists of 1 part cement, $2\frac{1}{2}$ parts sand, and 5 parts stone. A denser mixture may be secured by varying these proportions somewhat; that is, the maker may find that he secures a more compact block by using $2\frac{3}{4}$ parts sand and $4\frac{3}{4}$ parts stone; but a leaner mixture than $1 : 2\frac{1}{2} : 5$ is not to be recommended. In strength, this mixture will have a crushing resistance far beyond any load that it will ever have to support. Even a mixture of $1:3:6$ or $1:3\frac{1}{2}:7$ will be stronger than necessary to sustain any ordinary load. Such a mixture, however, would be porous and unsatisfactory in the wall of a building. Blocks, in being handled at the factory, carted to the building site, and in being

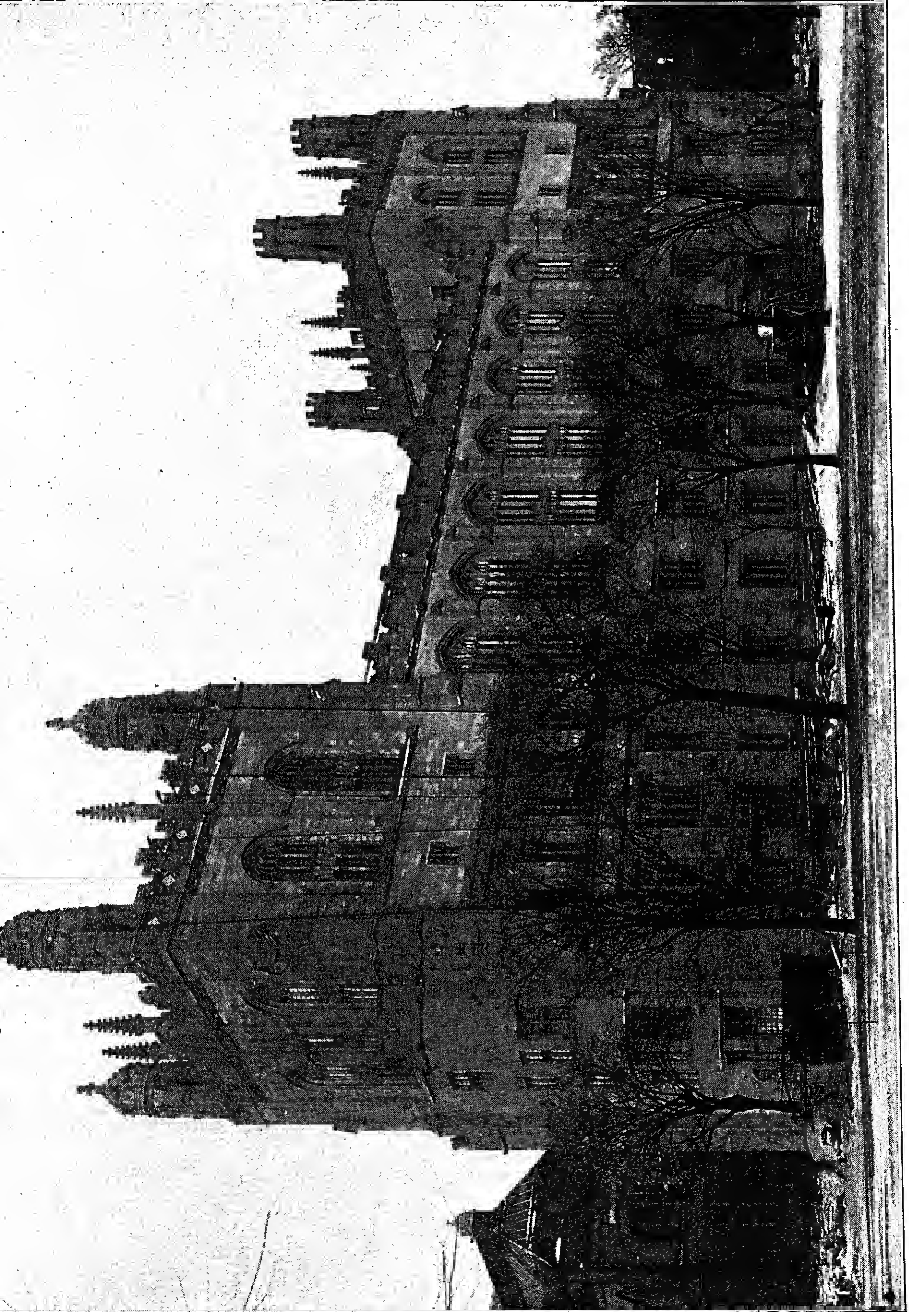
dling; and safety in this respect calls for a stronger block than is needed to bear the weight of a wall of a building. For a high-grade water-tight block, a 1:2:4 or a 1:2½:4 mixture is always used.

Amount of Water. Blocks made with dry concrete will be soft and weak, even if they are well sprinkled after being taken out of the forms. Blocks that are to be removed from the machine as soon as they are made will stick to the plates and sag out of shape, if the concrete is mixed too wet. Therefore there should be as much water as possible used, without causing the block to stick or sag out of shape when being removed from the molds. This amount of water is generally 8 to 9 per cent of the weight of the dry mixture. To secure uniform blocks in strength and color, the same amount of water should be used for each batch.

Mixing and Tamping. The concrete should be mixed in a batch mixer, although good results are obtained in hand-mixed concrete. The tamping is generally done with hand rammers. Pneumatic tampers, operated by an air compressor, are used successfully. Molding concrete by *pressure* is not successful unless the concrete is laid in comparatively thin layers.

Curing of Blocks. (a) *Air Curing.* The blocks are removed from the machine on a steel plate, on which they should remain for 24 hours. The blocks should be protected from the sun and dry winds for at least a week, and thoroughly sprinkled frequently. They should be at least four weeks old before they are placed in a wall. If they are built up in a wall while green, shrinkage cracks will be apt to occur in the joints.

(*b*) *Steam Curing.* Concrete blocks can be cured much more quickly in a steam chamber than in the open air. They should be left in the steam chamber for 48 hours at a pressure of 80 pounds per square inch. By this method of curing blocks they can be handled and used much quicker than when air cured. Their strength is then much higher than the air-cured blocks when six months old. When a large quantity of blocks are to be made, the steam curing is more economical than the air curing, even considering the much more expensive plant that is required. See Technologic Papers, Bureau of Standards, (U. S.) No. 5.



mixture is often used, generally consisting of 1 part cement to 2 parts sand. The penetration of water may be effectively prevented by this rich coat. Care must be taken to avoid a seam between the two mixtures.

Blocks are made with either a plane face or of various ornamental patterns, as tool-faced, paneled, rock-faced, etc. Coloring of the face is often desired. Mineral coloring, rather than chemical, should be used, as the chemical color may injure the concrete or fade.

Cost of Making. The following is quoted from a paper by N. F. Palmer, C. E.:

Blocks 8 by 9 by 32 inches; gang consisted of five workmen and a foreman; record for one hour, 30 blocks; general average for 10 hours, 200 blocks. The itemized cost was as follows:

1 foreman	@	$2.50	$ 2.50
5 helpers	@	2.00	10.00
13 bbls. cement	@	2.00	26.00
10 cu. yds. sand and gravel	@	1.00	10.00
Interest and depreciation on machine			2.00
			$50.50

This is the equivalent of $50.50 ÷ 200, or 25¼ cents per block; or, since the face of the block was 9 by 32 inches, or exactly 2 square feet, the equivalent of 12.6 cents per square foot of an 8-inch wall.

Another illustration, quoted from Gillette, for a 10-inch wall, was itemized as follows, *for each square foot* of wall:

Sand	2.0 cents
Cement @ $1.60 per barrel	4.5 cents
Labor @ $1.83 per day	3.8 cents
Total per square foot	10.3 cents

This is apparently considerably cheaper than the first case, even after allowing for the fact that the second case does not provide for interest, depreciation on plant, etc., which in the first case is only 4 per cent of the total. This allowance of 4 per cent is probably too small.

CEMENTING MATERIALS

The principal cementing materials are *Common Lime*, *Hydraulic Lime*, *Pozzuolana*, *Natural Cement*, and *Portland Cement*. There are a few other varieties, but their use is so limited that they need not be considered here.

marble, a limestone usually contains other substances—perhaps up to 10 per cent of silica, alumina, magnesia, etc. The process of burning drives off the carbonic acid and leaves the protoxide of calcium. This is the lime of commerce; and to preserve it from deterioration, it must be kept dry and even protected from a free circulation of air. When exposed freely to the air for a long period, it will become *air-slaked;* that is, it will absorb both moisture and carbonic acid from the air, and will lose its ability to harden. The first step in using common lime is to combine it with water, which it absorbs readily so that its volume is increased to $2\frac{1}{2}$ to $3\frac{1}{2}$ times what it was before. Its weight is at the same time increased about one-fourth; and the mass, which consisted originally of large lumps with some powder, is reduced to an unctuous mass of smooth paste. The lime is then called *slaked lime,* the process of slaking being accompanied by the development of great heat. The purer the lime, the greater the development of heat and the greater the expansion in volume. It is soluble in water which is not already "hard", or which does not already contain considerable lime in solution. A good lime will make a smooth paste with only a very small percentage (less than 10 per cent) of foreign matter or clinker. By such simple means a lime may be readily tested.

The hardening of common lime mortar is due to the formation of a carbonate of lime (substantially the original condition of the stone) by the absorption from the atmosphere of carbonic oxide. This will penetrate for a considerable depth in course of time; but instances are common in which masonry has been torn down after having been erected many years, and the lime mortar in the interior of the mass has been found still soft and unset, since it was hermetically cut off from the carbonic oxide of the atmosphere. For the same reason, common lime mortar will not harden under water and, therefore, it is utterly useless to employ it for work under water or for large masses of masonry.

When the qualities of slaking and expansion are not realized or are obtained only very imperfectly, the lime is called *lean* or *poor* (rather than *fat*) and its value is less and less, until it is perhaps worthless for use in making mortar, or for any other use except as

net.

Hydraulic Lime. This is derived from limestones containing about 10 to 20 per cent of clay or silica, which is intimately mixed with the carbonate of lime in the structure of the stone. During the process of burning, some of the lime combines with the clay (or the silica) so as to form the aluminate or silicate of lime. The excess of lime becomes quicklime as before. During the process of slaking, which should be done by mere *sprinkling*, the lime having been intimately mixed with the clay or silica, the expansion of the lime completely disintegrates the whole mass. This slaking is done by the manufacturer. The lime having a much greater avidity for the water than the aluminate or the silicate, the small amount of water used in the slaking is absorbed entirely by the lime, and the aluminate or the silicate is not affected. The setting of hydraulic lime appears to be due to the crystallizing of the aluminate and silicate; and since this will be accomplished even when the masonry is under water, it receives from this property its name of *hydraulic lime*. It is used but little in this country, and is all imported.

Pozzuolana or Slag Cement. Pozzuolana is a form of cementing material which has been somewhat in use since very ancient times. Apparently it was first made from the lava from the volcano Vesuvius, the lava being picked up at Pozzuoli, a village near the base of the volcano. It consists of a combination of silica and alumina, which is mixed with common lime. Its chemical composition is therefore not very unlike that of hydraulic lime. It also possesses the ability to harden under water. Its use is very limited, and its strength and hardness relatively small, when compared with that of Portland cement. It should never be used where it will be exposed for a long time to dry air, even after it has thoroughly set. It appears to withstand the action of sea water somewhat better than Portland cement; and hence it is sometimes used instead of Portland cement as the cementing material for large masses of masonry or concrete which are to be deposited in sea water, when the strength of the cement is a comparatively minor consideration. Artificial pozzuolana is sometimes made by grinding up blast-furnace slag which has been found by chemical analysis to have the correct chemical composition.

Natural Cement. Natural cement is obtained by burning an argillaceous or a magnesian limestone which happens to have the proper chemical composition. The resulting clinker is then finely ground and is at once ready for use. Such cement was formerly and is still commonly called *Rosendale cement,* owing to its having been produced first in Rosendale, Ulster County, New York. A very large part of the natural cement now produced in this country comes from Ulster County, New York, or from near Louisville, Kentucky. Cement rock from which natural cement can be made is now found widely scattered over the country.

In Europe, the name *Roman cement* is applied to substantially the same kind of product. Since the cement is made wholly from the rock just as it is taken out of the quarry, and also since it is calcined at a much lower temperature than that employed in making Portland cement it is considerably cheaper than Portland cement. On the other hand, its strength is considerably less than that of Portland cement and the time of setting is much quicker. Sometimes this quickness of setting is a very important point—as, for instance, when it is desired to obtain a concrete which shall attain considerable hardness very quickly. On the other hand, the quickness of setting may be a serious disadvantage, because it may not allow sufficient time to finish the concrete work satisfactorily without disturbing the mortar which has already taken an initial set. Natural cement is only used on account of its cheapness, and especially when the cement is not required to have very great strength. The disadvantage due to its quick setting (when it is a disadvantage) may be somewhat overcome by the use of a small percentage of lime, when mixing up the mortar.

It is not always admitted, at least in the advertisements, that a given brand of cement is a natural cement; and the engineer must therefore be on his guard, in buying a cement, to know whether it is a quick-setting natural cement of comparatively low strength or a true Portland cement.

Portland Cement. Portland cement consists of the product of burning and grinding an artificial mixture of carbonate of lime and clay or slag, the mixture being very carefully proportioned so that the ingredients shall have very nearly the fixed ratio which experience has demonstrated to give the best results.

of exactly uniform composition, could be found, Portland cement could be made from it, simply by burning and grinding. For good results, however, the composition of the raw material must be *exact*, and the proportion of carbonate of lime in it must not vary even by one per cent. No natural deposit of rock of exactly this correct and unvarying composition is known or likely ever to be found; therefore Portland cement is always made from an artificial mixture, usually, if free from organic matter, containing about 75 per cent carbonate of lime and 25 per cent clay."—S. B. NEWBERRY, in Taylor and Thompson's "Concrete, Plain and Reinforced."

As before stated, Portland cement is stronger than natural cement; it sets more slowly, which is frequently a matter of great advantage, and yet its rate of setting is seldom so slow that it is a disadvantage. Although the cost is usually greater than that of natural cement, yet improved methods of manufacture have reduced its cost so that it is now usually employed for all high-grade work where high ultimate strength is an important consideration.

In a general way, it may be said that the characteristics of Portland cement on which its value as a material to be used in construction work chiefly depends may be briefly indicated as follows:

When the cement is mixed with water and allowed to set, it should harden in a few hours, and should develop a considerable proportion of its ultimate strength in a few days. It should also possess the quality of permanency, so that no material change in form or volume will take place on account of its inherent qualities or as the result of exterior agencies. There is always found to be more or less of shrinkage in the volume of cement and concrete during the process of setting and hardening; but with any cement of really good quality, this shrinkage is not so great as to prove objectionable. Another very important characteristic is that the cement shall not lose its strength with age. Although some long-time tests of cement have apparently indicated a slight decrease in the strength of cement after the first year or so, this decrease is nevertheless so slight that it need not affect the design of concrete, even assuming the accuracy of the general statement.

To insure absolute dependence on the strength and durability of any cement which it is proposed to use in important structural work, it is essential that the qualities of the cement be determined by thorough tests.

All cement should be tested. On large operations a testing laboratory can be fitted up and all cement tested at the site of the operation. On smaller jobs the tests are generally made by professional laboratories. The cost of these tests is small. The professional laboratories keep men at all the big cement plants so that they can secure samples when the shipments are being made. Often by the time that the cement is received at the job and unloaded the report of the seven-day test will be also received at the work.

Standard Tests. The following method of testing cement is taken from the "Final Report on Tests of Cement" made to the American Society of Civil Engineers by a committee appointed to investigate and report on that subject, and is copied here from the proceedings of that Society by permission of their secretary, Charles Warren Hunt. The report on "Methods of Testing Cement" is printed in Vol. LXXV and the "Standard Specifications" is printed in the February, 1913, number of the proceedings of that society.

Methods for Testing Cement*

Sampling

1. *Selection of Sample.* The selection of samples for testing should be left to the engineer. The number of packages sampled and the quantity taken from each package will depend on the importance of the work and the facilities for making the tests.

2. The samples should fairly represent the material. When the amount to be tested is small it is recommended that one barrel in ten be sampled; when the amount is large it may be impracticable to take samples from more than one barrel in thirty or fifty. When the samples are taken from bins at the mill one for each fifty to two hundred barrels will suffice.

3. Samples should be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material; the use of this sieve is also effective to obtain a thorough mixing of the samples when this is desired. To determine the acceptance or rejection of cement it is preferable, when time permits, to test the samples separately. Tests to determine the general characteristics of a cement, extending over a long period, may be made with mixed samples.

*Accompanying Final Report of Special Committee on Uniform Tests of Cement, dated January 17th, 1912.

4. *Method of Sampling.* Cement in barrels should be sampled through a hole made in the head, or in one of the staves midway between the heads, by means of an auger or a sampling iron similar to that used by sugar inspectors; if in bags, the sample should be taken from surface to center; cement in bins should be sampled in such a manner as to represent fairly the contents of the bin. Sampling from bins is not recommended if the method of manufacture is such that ingredients of any kind are added to the cement subsequently.

Chemical Analysis

5. *Significance.* Chemical analysis may serve to detect adulteration of cement with inert material, such as slag or ground limestone, if in considerable amount. It is useful in determining whether certain constituents, such as magnesia and sulphuric anhydride, are present in inadmissible proportions.

6. The determination of the principal constituents of cement, silica, alumina, iron oxide, and lime is not conclusive as an indication of quality. Faulty cement results more frequently from imperfect preparation of the raw material or defective burning than from incorrect proportions. Cement made from material ground very fine and thoroughly burned may contain much more lime than the amount usually present, and still be perfectly sound. On the other hand, cements low in lime may, on account of careless preparation of the raw material, be of dangerous character. Furthermore, the composition of the product may be so greatly modified by the ash of the fuel used in burning as to affect in a great degree the significances of the results of analysis.

7. *Method.* The method to be followed should be that proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, reported in the *Journal* of the Society for Chemical Industry, Vol. 21, page 12, 1902; and published in *Engineering News*, Vol. 50, p. 60, 1903; and in *Engineering Record* Vol. 48, p. 49, 1903, and in addition thereto, the following:

The insoluble residue may be determined as follows: To a 1-gram sample of the cement are added 30 cu. cm. of water and 10 cu. cm. of concentrated hydrochloric acid, and then warmed until the effervescence ceases, and digested on a steam bath until dissolved. The residue is filtered, washed with hot water, and the filter paper and contents digested on the steam bath in a 5% solution of sodium carbonate. This residue is filtered, washed with hot water, then with hot hydrochloric acid, and finally with hot water, and then ignited at a red heat and weighed. The quantity so obtained is the insoluble residue.

very small, great care must be exercised in making the determination.

10. *Apparatus.* The determination of specific gravity should be made with a standardized Le Chatelier apparatus. This consists of a flask (*D*), Fig. 1, of about 120 cu. cm. capacity, the neck of which is about 20 cm. long; in the middle of this neck is a bulb (*C*),

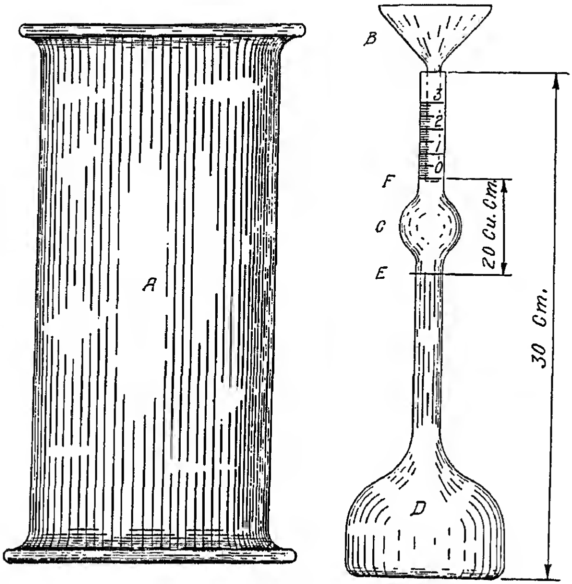


Fig. 1. Le Chatelier Apparatus for Determining Specific Gravity of Cement

above and below which are two marks (*F*) and (*E*); the volume between these two marks is 20 cu. cm. The neck has a diameter of about 9 mm., and is graduated into tenths of cubic centimeters above the mark (*F*).

11. Benzine (62° Baumé naphtha) or kerosene free from water should be used in making the determination.

12. *Method.* The flask is filled with either of these liquids to the lower mark (*E*), and 64 grams of cement, cooled to the temperature of the liquid, is slowly introduced through the funnel (*B*),

(the stem of which should be long enough to extend into the flask to the top of the bulb (C)), taking care that the cement does not adhere to the sides of the flask and that the funnel does not touch the liquid. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; this reading, plus 20 cu. cm. is the volume displaced by 64 grams of the cement.

13. The specific gravity is then obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement in grams}}{\text{Displaced volume in cubic centimeters}}$$

14. The flask, during the operation, is kept immersed in water in a jar (A), in order to avoid variations in the temperature of the liquid in the flask, which should not exceed $\frac{1}{2}°$ C. The results of repeated tests should agree within 0.01. The determination of specific gravity should be made on the cement as received; if it should fall below 3.10, a second determination should be made after igniting the sample at a low red heat in the following manner: One-half gram of cement is heated in a weighed platinum crucible, with cover, for 5 minutes with a Bunsen burner (starting with a low flame and gradually increasing to its full height) and then heating for 15 minutes with a blast lamp; the difference between the weight after cooling and the original weight is the loss on ignition. The temperature should not exceed 900° C., and the ignition should preferably be made in a muffle.

15. The apparatus may be cleaned in the following manner: The flask is inverted and shaken vertically, until the liquid flows freely, and then held in a vertical position until empty; any traces of cement remaining can be removed by pouring into the flask a small quantity of clean benzine or kerosene, and repeating the operation.

Fineness

16. *Significance.* It is generally accepted that the coarser particles in cement are practically inert and that only the extremely fine powder possesses cementing qualities. The more finely cement is pulverized, other conditions being the same, the more sand it will carry and so produce a mortar of a given strength.

17. *Apparatus.* The fineness of a sample of cement is determined by weighing the residue retained on certain sieves. Those known as No. 100 and No. 200, having approximately 100 and 200 wires per linear inch, respectively, should be used. They should be at least 8 inches in diameter. The wire cloth should be of brass wire and should conform to the following requirements:

No. of Sieve	Diameter of Wire	Meshes per Linear Inch	
		Warp	Woof
100	0.0042 to 0.0048 in.	95 to 101	93 to 103
200	0.0021 to 0.0023 in.	.192 to 203	190 to 205

The meshes in any smaller space, down to 0.25 inch, should be proportional in number.

18. *Method.* The test should be made with 50 grams of cement, dried at a temperature of 100° C. (212° F.).

19. The cement is placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position and moved forward and backward about 200 times per minute, at the same time striking the side gently, on the up stroke, against the palm of the other hand. The operation is continued until not more than 0.05 gram will pass through in one minute. The residue is weighed, then placed on the No. 100 sieve, and the operation repeated. The work may be expedited by placing in the sieve a few large steel shot, which should be removed before the final one minute of sieving. The sieves should be thoroughly dry and clean.

Normal Consistency

20 *Significance.* The use of a proper percentage of water in making pastes* and mortars for the various tests is exceedingly important and vitally affects the results obtained.

21. The amount of water, expressed in percentage by weight of the dry cement required to produce a paste of plasticity desired, termed "normal consistency", should be determined with the Vicat apparatus in the following manner:

22. *Apparatus.* This consists of a frame (*A*), Fig. 2, bearing a movable rod (*B*), weighing 300 grams, one end (*C*) being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle (*D*), 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in any desired position by a screw (*E*), and has midway between the ends a mark (*F*) which moves under a scale (graduated to millimeters) attached to the frame (*A*). The paste is held by a conical, hard-rubber ring (*G*), 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate (*H*) about 10 cm. square.

23. *Method.* In making the determination, the same quantity of cement as will be used subsequently for each batch in making the

*The term "paste" is used in this report to designate a mixture of cement and water, and the word "mortar" to designate a mixture of cement, sand, and water.

test pieces, but not less than 500 grams, with a measured quantity of water, is kneaded into a paste, as described in paragraph 45, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 inches apart; the ball resting in the palm of one hand is pressed into the larger end of the rubber ring held in the other hand, completely

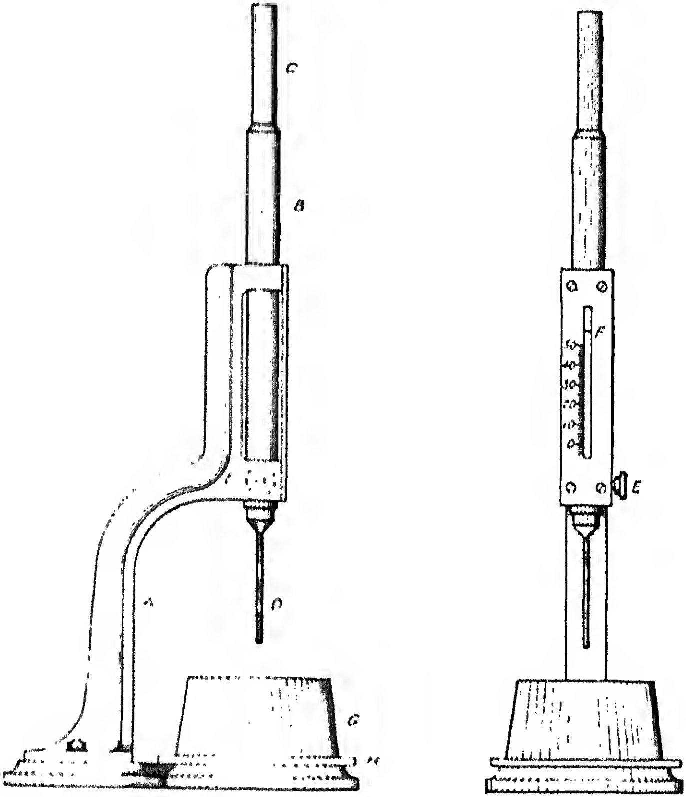


Fig. 2. Vicat Apparatus for Testing Normal Consistency of Cement

filling the ring with paste; the excess at the larger end is then removed by a single movement of the palm of the hand; the ring is then placed on its larger end on a glass plate and the excess paste at the smaller end is sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care must be taken not to compress the paste. The paste confined in the ring, resting on the plate, is placed under the rod, the larger end of which is brought in contact with the surface of the paste; the scale is then read and the rod quickly released.

24. The paste is of normal consistency when the rod settles to a point 10 mm. below the original surface in one-half minute after being released. The apparatus must be free from all vibrations during the test.

25. Trial pastes are made with varying percentages of water until the normal consistency is obtained.

26. Having determined the percentage of water required to produce a paste of normal consistency, the percentage required for a mortar containing by weight one part of cement to three parts of standard Ottawa sand is obtained from Table II, the amount being a percentage of the combined weight of the cement and sand.

TABLE II

Percentage of Water for Standard Mortars

Neat	One cement, three standard Ottawa sand	Neat	One cement, three standard Ottawa sand	Neat	One cement, three standard Ottawa sand
15	8.0	23	9.3	31	10.7
16	8.2	24	9.5	32	10.8
17	8.3	25	9.7	33	11.0
18	8.5	26	9.8	34	11.2
19	8.7	27	10.0	35	11.3
20	8.8	28	10.2	36	11.5
21	9.0	29	10.3	37	11.7
22	9.2	30	10.5	38	11.8

Time of Setting

27. *Significance.* The object of this test is to determine the time which elapses from the moment water is added until the paste ceases to be plastic (called the "initial set"), and also the time until it acquires a certain degree of hardness (called the "final set" or "hard set"). The former is the more important, since, with the commencement of setting, the process of crystallization begins. As a disturbance of this process may produce a loss of strength, it is desirable to complete the operation of mixing or molding or incorporating the mortar into the work before the cement begins to set.

28 *Apparatus.* The initial and final set should be determined with the Vicat apparatus described in paragraph 22.

29. *Method.* A paste of normal consistency is molded in the hard-rubber ring, as described in paragraph 23, and placed under the rod (*B*), the smaller end of which is then carefully brought in contact with the surface of the paste, and the rod quickly released.

30. The initial set is said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate; and the final set, when the needle does not sink visibly into the paste.

31. The test pieces should be kept in moist air during the test; this may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth; the cloth to be kept from contact with them by means of a wire screen; or they may be stored in a moist box or closet.

32. Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration.

33. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is, therefore, only approximate.

Standard Sand

34. The sand to be used should be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve, and retained on a No. 30 sieve. The sieves should be at least 8 inches in diameter; the wire cloth should be of brass wire and should conform to the following requirements:

No. of Sieve	Diameter of Wire	Meshes per Linear Inch	
		Warp	Woof
20	0.016 to 0.017 in.	19.5 to 20.5	19 to 21
30	0.011 to 0.012 in.	29.5 to 30.5	28.5 to 31.5

Sand which has passed the No. 20 sieve is standard when not more than 5 grams passes the No. 30 sieve in one minute of continuous sifting of a 500-gram sample.*

Form of Test Pieces

35. For tensile tests the form of test piece shown in Fig. 3 should be used.

36. For compressive tests, 2-inch cubes should be used.

Molds

37. The molds should be of brass, bronze, or other noncorrodible material, and should have sufficient metal in the sides to prevent spreading during molding.

38. Molds may be either single or gang molds. The latter are preferred by many. If used, the types shown in Fig. 4 are recommended.

*This sand may be obtained from the Ottawa Silica Company at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.

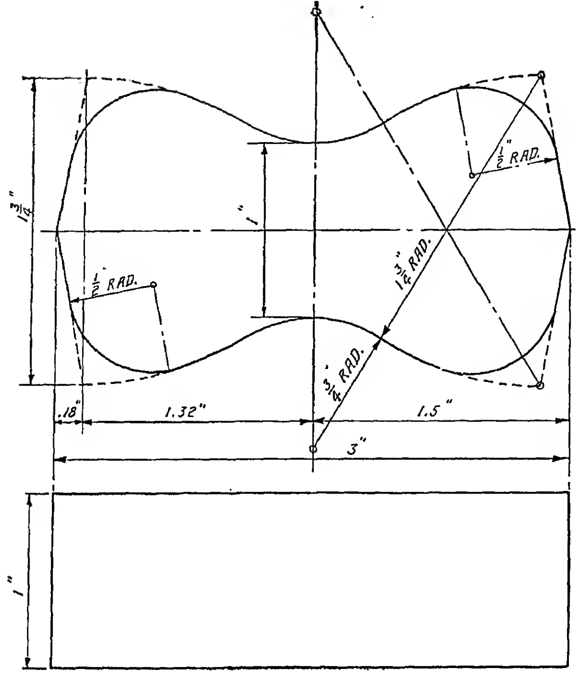


Fig. 3. Diagram Showing Form and Dimensions of Standard Cement Briquette to be Used for Testing

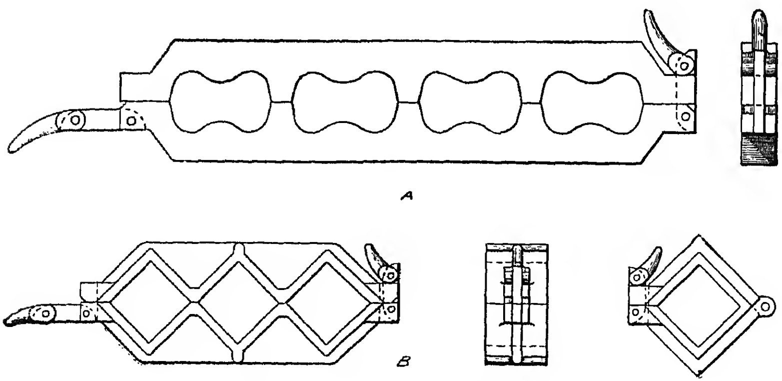


Fig. 4. Types of Briquette Molds

Mixing

40. The proportions of sand and cement should be stated by weight; the quantity of water should be stated as a percentage by weight of the dry material.

41. The metric system is recommended because of the convenient relation of the gram and the cubic centimeter.

42. The temperature of the room and of the mixing water should be maintained as nearly as practicable at 21° C. (70° F.)

43. The quantity of material to be mixed at one time depends on the number of test pieces to be made; 1000 grams is a convenient quantity to mix by hand methods.

44. The Committee has investigated the various mechanical mixing machines thus far devised, but cannot recommend any of them, for the following reasons: (1) the tendency of most cement is to "ball up" in the machine, thereby preventing working it into a homogeneous paste; (2) there are no means of ascertaining when the mixing is complete without stopping the machine; and (3) it is difficult to keep the machine clean.

45. *Method.* The material is weighed, placed on a non-absorbent surface (preferably plate glass), thoroughly mixed dry, if sand be used, and a crater formed in the center, into which the proper percentage of clean water is poured; the material on the outer edge is turned into the center by the aid of a trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for one minute. During the operation the hands should be protected by rubber gloves.

Molding

46. The Committee has not been able to secure satisfactory results with existing molding machines; the operation of machine molding is very slow, and is not practicable with pastes or mortars containing as large percentages of water as herein recommended.

47. *Method.* Immediately after mixing, the paste or mortar is placed in the molds with the hands, pressed in firmly with the fingers, and smoothed off with a trowel without ramming. The material should be heaped above the mold, and, in smoothing off, the trowel should be drawn over the mold in such a manner as to exert a moderate pressure on the material. The mold should then be turned over and the operation of heaping and smoothing off repeated.

48. A check on the uniformity of mixing and molding may be afforded by weighing the test pieces on removal from the moist

closet; test pieces from any sample which vary in any more than 3% from the average should not be considered.

Storage of the Test Pieces

49. During the first 24 hours after molding, the test pieces should be kept in moist air to prevent drying.

50. Two methods are in common use to prevent drying: (1) covering the test pieces with a damp cloth, and (2) placing them in a moist closet. The use of the damp cloth, as usually carried out, is

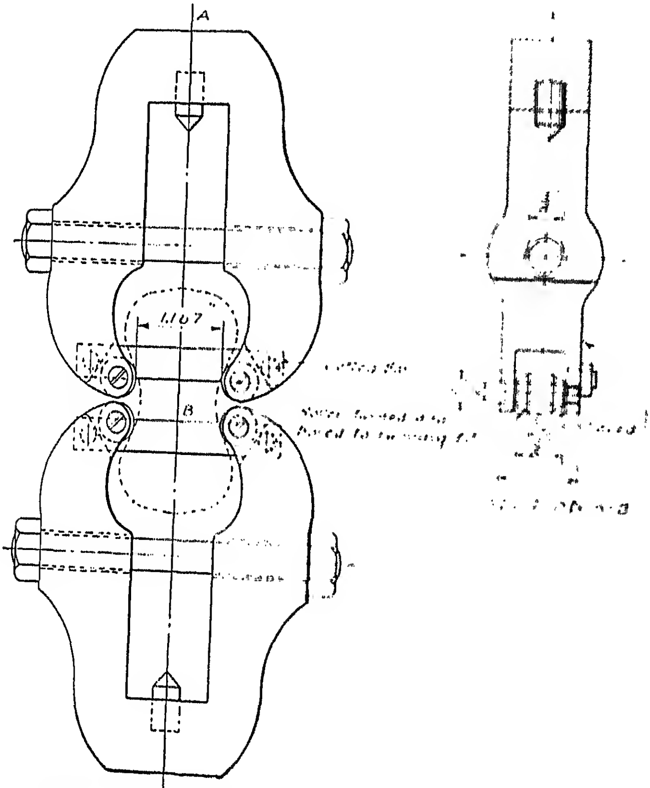
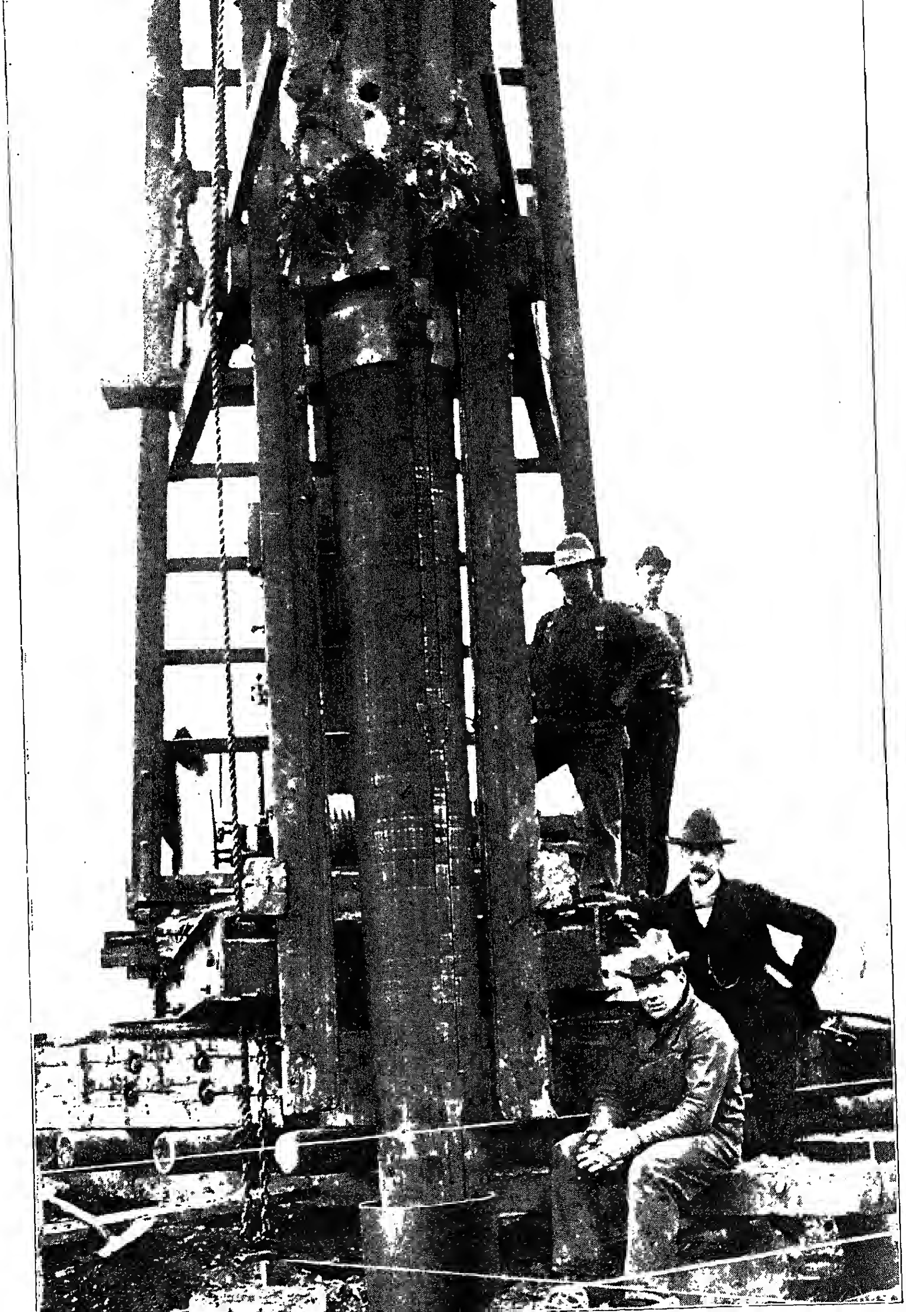


Fig. 5. Diagram Showing Construction of Metal Clip for Holding Cement Briquettes under Test

objectionable, because the cloth may dry out unequally and in consequence the test pieces will not all be subjected to the same degree of moisture. This defect may be remedied to some extent by immersing the edges of the cloth in water; contact between the cloth and the test pieces should be prevented by means of a wire screen, or some similar arrangement. A moist closet is



in securing uniformly moist air, and is so easily devised and so inexpensive, that the use of the damp cloth should be abandoned.

51. A moist closet consists of a soapstone or slate box, or a wood box lined with metal, the interior surface being covered with felt or broad wicking kept wet, the bottom of the box being kept covered with water. The interior of the box is provided with glass shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily.

52. After 24 hours in moist air, the pieces to be tested after longer periods should be immersed in water in storage tanks or pans made of non-corrodible material.

53. The air and water in the moist closet and the water in the storage tanks should be maintained as nearly as practicable at 21° C. (70° F.).

Tensile Strength

54. The tests may be made with any standard machine.

55. The clip is shown in Fig. 5. It must be made accurately, the pins and rollers turned, and the rollers bored slightly larger than the pins, so as to turn easily. There should be a slight clearance at each end of the roller, and the pins should be kept properly lubricated and free from grit. The clips should be used without cushioning at the points of contact.

56. Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the test pieces in the testing machine, as cross strains, produced by imperfect centering, tend to lower the breaking strength. The load should not be applied too suddenly, as it may produce vibration, the shock from which often causes the test piece to break before the ultimate strength is reached. The bearing surfaces of the clips and test pieces must be kept free from grains of sand or dirt, which would prevent a good bearing. The load should be applied at the rate of 600 pounds per minute. The average of the results of the test pieces from each sample should be taken as the test of the sample. Test pieces which do not break within $\frac{1}{4}$ inch of the center, or are otherwise manifestly faulty, should be excluded in determining average results.

Compressive Strength

57. The tests may be made with any machine provided with means for so applying the load that the line of pressure is along the axis of the test piece. A ball-bearing block for this purpose is shown in Fig. 6. Some appliance should be provided to facilitate placing the axis of the test piece exactly in line with the center of

58. The test piece should be placed in the testing machine, with a piece of heavy blotting paper on each of the crushing faces, which should be those that were in contact with the mold.

Constancy of Volume

59. *Significance.* The object is to detect those qualities which tend to destroy the strength and durability of a cement. Under normal conditions these defects will in some cases develop quickly, and

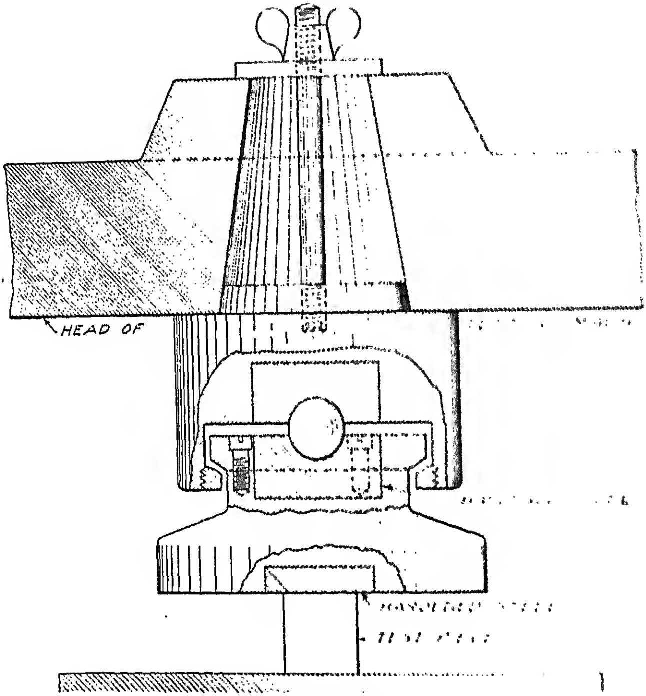


Fig. 6. Part Section of Head of Machine for Making Compression Tests on Cement Blocks

in other cases may not develop for a considerable time. Since the detection of these destructive qualities before using the cement in construction is essential, tests are made not only under normal conditions but under artificial conditions created to hasten the development of these defects. Tests may, therefore, be divided into two classes: (1) normal tests, made in either air or water maintained, as nearly as practicable, at 21° C. (70° F.); and (2) accelerated tests, made in air, steam, or water, at temperature of 45° C. (113° F.) and upward. The Committee recommends that these tests be

60. *Methods.* Pats, about 3 inches in diameter, $\frac{1}{2}$ inch thick at the center, and tapering to a thin edge, should be made on clean glass plates (about 4 inches square) from cement paste of normal consistency, and stored in a moist closet for 24 hours.

61. *Normal Tests.* After 24 hours in the moist closet, a pat is immersed in water for 28 days and observed at intervals. A similar pat, after 24 hours in the moist closet, is exposed to the air for 28 days or more and observed at intervals.

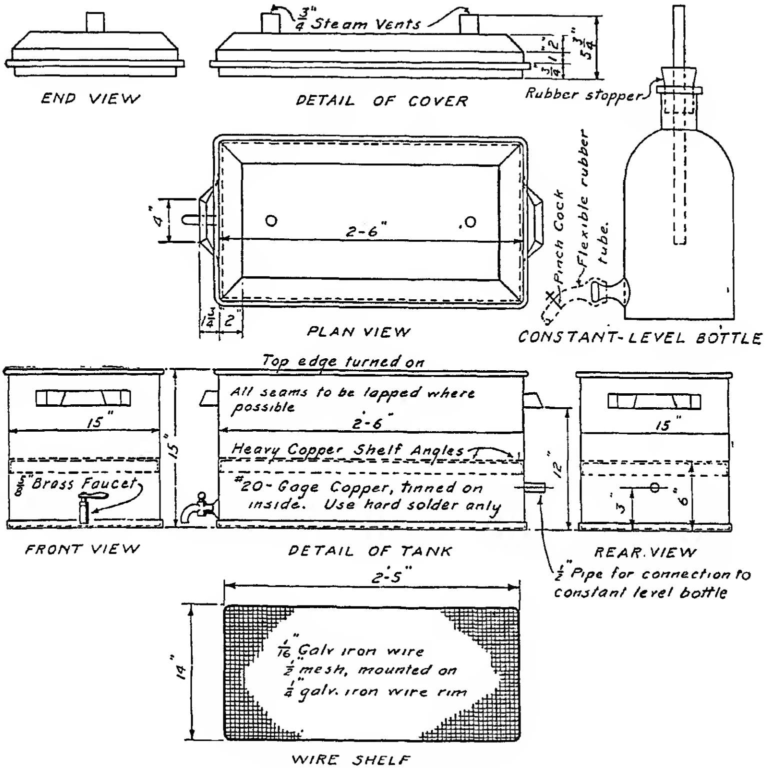


Fig. 7. Details of Apparatus for Making Accelerated Tests on Cement Blocks

62. *Accelerated Test.* After 24 hours in the moist closet, a pat is placed in an atmosphere of steam, upon a wire screen 1 inch above boiling water, for 5 hours. The apparatus should be so constructed that the steam will escape freely and atmospheric pressure be maintained. Since the type of apparatus used has a great influence on the results, the arrangement shown in Fig. 7 is recommended.

63. Pats which remain firm and hard and show no signs of

cracking, distortion, or disintegration are said to be "of constant volume" or "sound".

64. Should the pat leave the plate, distortion may be detected best with a straightedge applied to the surface which was in contact with the plate.

65. In the present state of our knowledge it cannot be said that a cement which fails to pass the accelerated test will prove defective in the work; nor can a cement be considered entirely safe simply because it has passed these tests.

George S. Webster, *Chairman.*
Richard L. Humphrey, *Secretary.*
W. B. W. Howe,
F. H. Lewis,
S. B. Newberry,
Alfred Noble,
Clifford Richardson,
L. C. Sabin,
George F. Swain.

Standard Cement Specifications*

General Observations

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY

3. Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS

4. The sieves should be kept thoroughly dry.

TIME OF SETTING

5. Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities vitally affect the rate of setting.

*Adopted August 16, 1909, by the American Society for Testing Materials.

6. The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

7. In making the pats, the greatest care should be exercised to avoid initial strains due to molding or to too rapid drying-out during the first 24 hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

8. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement, however, may be held for 28 days, and a retest made at the end of that period, using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

GENERAL CONDITIONS

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the contractor, and a period of at least 12 days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages, with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the 7-day requirements may be held awaiting the results of the 28-day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society on January 17th, 1912, with all subsequent amendments thereto.

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement

11. *Definition.* This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS

12. It shall leave by weight a residue of not more than 10% on the No. 100, and 30% on the No. 200 sieve.

TIME OF SETTING

13. It shall not develop inital set in less than 10 minutes, and shallnot develop hard set in less than 30 minutes, or more than 3 hours.

TENSILE STRENGTH

14. The minimum requirements for tensile strength for briquettes 1 square inch in cross section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Neat Cement

AGE	STRENGTH
24 hours in moist air	75 lb.
7 days (1 day in moist air, 6 days in water)	150 lb.
28 days (1 day in moist air, 27 days in water)	250 lb.

One Part Cement, Three Parts Standard Ottawa Sand

7 days (1 day in moist air, 6 days in water)	50 lb.
28 days (1 day in moist air, 27 days in water)	125 lb.

CONSTANCY OF VOLUME

15. Pats of neat cement about 3 inches in diameter, ½ inch thick at the center, tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° F. as practicable.

16. These pats are observed at intervals for at least 28 days, and, to pass the tests satisfactorily, should remain firm and hard and show no signs of distortion, checking, cracking, or disintegrating.

Portland Cement

17. *Definition.* This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3% has been made subsequent to calcination.

SPECIFIC GRAVITY

18. The specific gravity of cement shall be not less than 3.10. Should the test of cement as received fall below this requirement, a second test may be made on a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4 per cent.

FINENESS

19. It shall leave by weight a residue of not more than 8% on the No. 100, and not more than 25% on the No. 200 sieve.

TIME OF SETTING

20. It shall not develop initial set in less than 30 minutes; and must develop hard set in not less than 1 hour, nor more than 10 hours.

TENSILE STRENGTH

21. The minimum requirements for tensile strength for briquettes 1 square inch in cross section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Neat Cement

AGE	STRENGTH
24 hours in moist air	175 lb.
7 days (1 day in moist air, 6 days in water)	500 lb.
28 days (1 day in moist air, 27 days in water)	600 lb.

One Part Cement, Three Parts Standard Ottawa Sand

7 days (1 day in moist air, 6 days in water)	200 lb.
28 days (1 day in moist air, 27 days in water)	275 lb.

CONSTANCY OF VOLUME

22. Pats of neat cement about 3 inches in diameter, ½ inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for 5 hours.

23. These pats, to pass the requirements satisfactorily, shall remain firm and hard, and show no signs of distortion, checking, cracking, or disintegrating.

SULPHURIC ACID AND MAGNESIA

24. The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO).

in Fig. 8. A reservoir contains a supply of shot, which falls through the pipe closed by means of a valve at the bottom. The briquette is carefully placed between the clips, as shown in the figure, and the

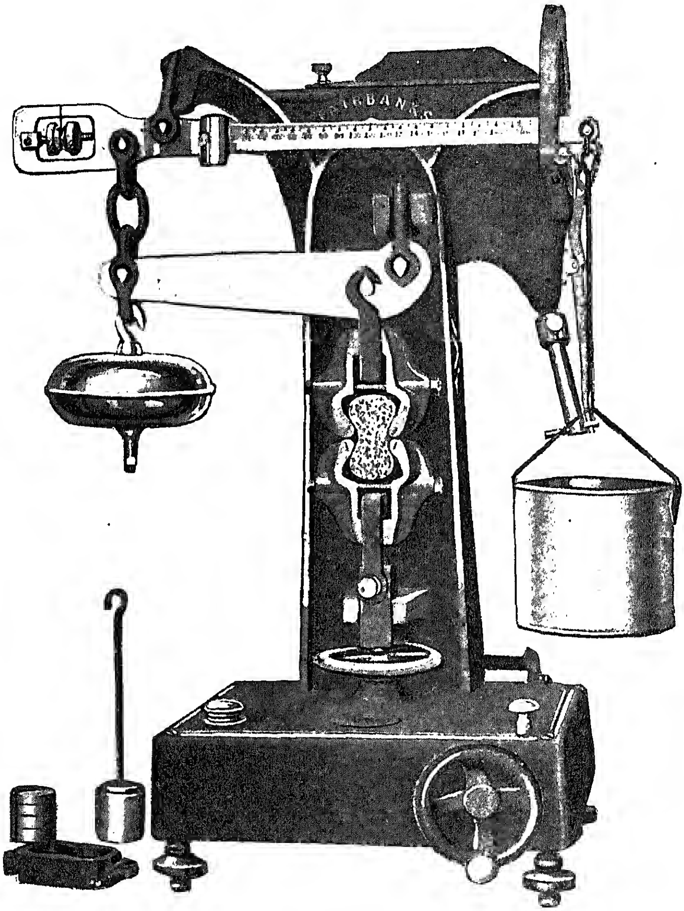


Fig. 8. Cement Testing Scales with Briquette in Position
Courtesy of Fairbanks, Morse and Company

wheel below is turned until the indicators are in line. A hook lever is moved so that a screw worm is engaged with its gear. Then the valve of the shot reservoir is opened so as to allow the shot to run into the cup, a small valve regulating the flow of shot into

the cup. Better results will be obtained by allowing the shot to run slowly into the cup. The crank is then turned with just sufficient speed so that the scale beam is held in position until the briquette is broken. Upon the breaking of the briquette, the scale beam falls, and automatically closes the valve. The weight of the shot in the cup then indicates, according to some definite ratio, the stress required to break the briquette.

SAND

Sand is a constituent part of mortar and concrete. The strength of the masonry is dependent to a considerable extent on the qualities of the sand and it is therefore important that the desirable and the defective qualities should be understood, and that these qualities be always investigated as thoroughly as are the qualities of the cement used. There have been many failures of structures due to the use of poor sand.

Object. Sand is required in mortar or concrete for economy and to prevent the excessive cracking that would take place in neat lime or cement without the use of sand. Mortar made without sand would be expensive and the neat lime or cement would crack so badly that the increased strength, due to the neat paste, would be of little value, if any, on account of it contracting and cracking very badly.

Essential Qualities. The word "sand" as used above is intended as a generic term to apply to any finely divided material which will not injuriously affect the cement or lime, and which is not subject to disintegration or decay. Sand is almost the only material which is sufficiently cheap and which will fulfil these requirements, although stone screenings (the finest material coming from a stone crusher), powdered slag, and even coal dust have occasionally been used as substitutes. Specifications usually demand that the sand shall be "sharp, clean, and coarse", and such terms have been repeated so often that they are accepted as standard, notwithstanding the frequent demonstration that modifications of these terms are not only desirable but also economical. These words also ignore other qualities which should be considered, especially when deciding between two or more different sources of sand supply.

Geological Character. Quartz sand is the most durable and unchangeable. Sands which consist largely of grains of feldspar, mica, hornblende, etc., which will decompose upon prolonged exposure to the atmosphere, are less desirable than quartz, although, after being made up into the mortar, they are virtually protected against further decomposition.

Coarseness. A mixture of coarse and fine grains, with the coarse grains predominating, is found very satisfactory, as it makes a denser and stronger concrete with a less amount of eement than when eoarse-grained sand is used with the same proportion of cement. The small grains of sand fill the voids caused by the coarse grains so that there is not so great a volume of voids to be filled by the cement. The sharpness of sand can be determined approximately by rubbing a few grains in the hand or by crushing it near the ear and noting if a grating sound is produced; but an examination through a small lens is better.

Sharpness. Experiments have shown that round grains of sand have less voids than angular ones, and that water-worn sands have from 3 to 5 per cent less voids than corresponding sharp grains. In many parts of the country where it is impossible, except at a great expense, to obtain the sharp sand, the round grain is used with very good results. Laboratory tests made under conditions as nearly as possible identical, show that the rounded-grain sand gives as good results as the sharp sand. In consequence of such tests, the requirement that sand shall be *sharp* is now considered useless by many engineers, especially when it leads to additional cost.

Cleanness. In all specifications for concrete work, is found the clause: "The sand shall be clean." This requirement is sometimes questioned, as experimenters have found that a small percentage of clay or loam often gives better results than when clean sand is used. "Lean" mortar may be improved by a small percentage of clay or loam, or by using dirty sand, for the fine material increases the density. In rich mortars, this fine material is not needed, as the cement furnishes all the fine material necessary and, if clay or loam or dirty sand were used, it might prove detrimental. Whether it is really a benefit or not, depends chiefly upon the richness of the concrete and the coarseness of the sand. Some idea of the cleanliness of sand may be obtained by placing it in the palm of

one hand and rubbing it with the fingers of the other. If the sand is dirty, it will badly discolor the palm of the hand. When it is found necessary to use dirty sand, the strength of the concrete should be tested.

Sand containing loam or earthy material is cleansed by washing with water, either in a machine specially designed for the purpose, or by agitating the sand with water in boxes provided with holes to permit the dirty water to flow away.

Very fine sand may be used alone, but it makes a weaker concrete than either coarse sand or coarse and fine sand mixed. A mortar consisting of very fine sand and cement will not be so dense as one of coarse sand and the same cement, although, when measured or weighed dry, both contain the same proportion of voids and solid matter. In a unit measure of fine sand, there are more grains than in a unit measure of coarse sand and, therefore, more points of contact. More water is required in gaging a mixture of fine sand and cement than in a mixture of coarse sand and the same cement. The water forms a film and separates the grains, thus producing a larger volume having less density.

The screenings of broken stone are sometimes used instead of sand. Tests frequently show a stronger concrete when screenings are used than when sand is used. This is perhaps due to the variable sizes of the screenings, which would have a less percentage of voids.

Percentage of Voids. As before stated, a mortar is strongest when composed of fine and coarse grains mixed in such proportion that the percentage of voids shall be the least. The simplest method of comparing two sands is to weigh a certain gross volume of each, the sand having been thoroughly shaken down. Assuming that the stone itself of each kind of sand has the same density, then the heavier volume of sand will have the least percentage of voids. The actual percentage of voids in packed sand may be approximately determined by measuring the volume of water which can be added to a given volume of packed sand. If the water is poured into the sand, it is quite certain that air will remain in the voids in the sand, which will not be dislodged by the water, and the apparent volume of voids will be *less* than the actual. The precise determination involves the measurement of the specific gravity of the stone of

which the sand is composed, and the percentage of fine silt in the sand, all of which is done with elaborate precautions. Ordinarily, such precise determinations are of little practical value, since the product of any one sand bank is quite variable. While it would be theoretically possible to mix fine and coarse sand, varying the ratio according to the varying coarseness of the grains as obtained from the sand pit, it is quite probable that an over-refinement in this particular would cost more than the possible saving is worth. Ordinarily, sand has from 28 to 40 per cent of voids. An experimental test of sand of various degrees of fineness, 12½ per cent of it passing a No. 100 sieve, showed only 22 per cent of voids; but such a value is of only theoretical interest.

BROKEN STONE

Classification of Stones. This term ordinarily signifies the product of a stone crusher or the result of hand-breaking by hammering large blocks of stone; but the term may also include *gravel*, described below.

The best, hardest, and most durable broken stone comes from the *trap rocks*, which are dark, heavy, close-grained rocks of igneous origin. The term *granite* is usually made to include not only true granite, but also gneiss, mica schist, syenite, etc. These are just as good for concrete work, and are usually less expensive. *Limestone* is suitable for some kinds of concrete work; but its strength is not so great as that of granite or trap rock, and it is more affected by a conflagration. *Conglomerate*, often called *pudding stone*, makes a very good concrete stone. The value of *sandstone* for concrete is very variable according to its texture. Some grades are very compact, hard, and tough, and make a good concrete; other grades are friable, and, like *shale* and *slate*, are practically unfit for use. *Gravel* consists of pebbles of various sizes, produced from stones which have been broken up and then worn smooth with rounded corners. The very fact that they have been exposed for indefinite periods to atmospheric disintegration and mechanical wear is a proof of the durability and mechanical strength of the stone.

Size of Stone and Its Uniformity. There is hardly any limitation to the size of stone which may be used in large blocks of massive concrete, since it is now frequently the custom to insert these large

stone. But the term *broken stone* should be confined to those pieces of a size which may be readily mixed up in a mass, as is done when mixing concrete; and this virtually limits the size to stones which will pass through a $2\frac{1}{2}$-inch ring. The lower limit in size is very indefinite, since the product of a stone crusher includes all sizes down to stone dust screenings, such as are substituted partially or entirely for sand, as previously noted. Practically the only use of broken stone in masonry construction is in the making of concrete; and, since one of the most essential features of good concrete construction is that the concrete shall have the greatest possible density, it is important to reduce the percentage of voids in the stone as much as possible. This percentage can be determined with sufficient accuracy for ordinary unimportant work, by the very simple method previously described for obtaining that percentage with sand—namely, by measuring how much water will be required to fill up the cavities in a given volume of dry stone. As before, such a simple determination is somewhat inexact, owing to the probability that bubbles of air will be retained in the stone which will reduce the percentage somewhat, and also because of the uncertainty involved as to whether the stone is previously dry or is saturated with water. Some engineers drop the stone slowly into the vessel containing the water, rather than pour the water into the vessel containing the stone, with the idea that the error due to the formation of air bubbles will be decreased by this method. The percentage of error, however, due to such causes, is far less than it is in a similar test of sand, and the error for ordinary work is too small to have any practical effect on the result.

Illustrative Example. A pail having a mean inside diameter of 10 inches and a height of 14 inches is filled with broken stone well shaken down; a similar pail filled with water to a depth of 8 inches is poured into the pail of stone until the water fills up all the cavities and is level with the top of the stone; there is still $2\frac{1}{4}$ inches depth of water in the pail. This means that a depth of $5\frac{3}{4}$ inches has been used to fill up the voids. The area of a 10-inch circle is 78.54 square inches and therefore the volume of the broken stone was $78.54 \times 14 = 1{,}099.56$ cubic inches. The volume of the water used to fill the pail was 78.54×5.75, or 451.6 cubic inches. This is 41 per cent

of the volume of the stone, and is in this case the [illegible]
The accuracy of the above computation depends largely on the accuracy of the measurement of the *mean inside diameter* of the pail. If the pail were truly cylindrical, there would be no inaccuracy. If the pail is flaring, the inaccuracy might be considerable; and if a precise value is desired, more accurate methods should be chosen to measure the volume of the stone and of the water.

Screened Stone Unnecessary. It is invariably found that unscreened stone or *the run of the crusher* has a far less percentage of voids than screened stone, and it is therefore not only an extra expense, but also an injury to the concrete, to specify that broken stone shall be screened before being used in concrete, unless, as described later, it is intended to mix definite proportions of several sizes of carefully screened broken stone. Since the proportion of large and small particles in the run of the crusher depends considerably upon the character of the stone which is being broken up, and perhaps to some extent on the crusher itself, these proportions should be tested at frequent intervals during the progress of the work; and the amount of sand to be added to make a good concrete should be determined by trial tests, so that the resulting percentage of voids shall be as small as it is practicable to make it. It is usually found that the percentage of voids in crusher-run granite is a little larger than in limestone or gravel. This gives a slight advantage to the limestone and gravel, which tends to compensate for the weakness of the limestone and the rounded corners of the gravel.

Broken stone is frequently sold by the ton, instead of by the cubic yard; but as its weight varies from 2200 to 3200 pounds per cubic yard, an engineer or contractor is uncertain as to how many cubic yards he is buying or how much it costs him per cubic yard, unless he is able to test the particular stone and obtain an average figure as to its weight per unit of volume.

Cinders. Cinders for concrete should be free from coal or soot. Usually a better mixture can be obtained by screening the fine stuff from the cinders and then mixing in a larger proportion of sand, than by using unscreened material, although, if the fine stuff is uniformly distributed through the mass, it may be used without screening, and a less proportion of sand used.

As shown later, the strength of cinder concrete is far less than

high compressive values are necessary. But on account of its very low cost compared with broken stone, especially under some conditions, it is used quite commonly for roofs, etc., on which the loads are comparatively small.

One possible objection to the use of cinders lies in the fact that they frequently contain sulphur and other chemicals which may produce corrosion of the reinforcing steel. In any structure where the strength of the concrete is a matter of importance, cinders should not be used without a thorough inspection, and even then the unit compressive values allowed should be at a very low figure.

MORTAR

Kinds of Mortar. The term *mortar* is usually applied to the mixture of sand and cementing material which is placed between the large stones of a stone structure, although the term might also be properly applied to the matrix of the concrete in which broken stone is embedded. The object of the mortar is to furnish a cushion for the stones above it, which, as far as possible, distributes the pressure uniformly and relieves the stones of transverse stresses and also from the concentrated crushing pressures to which the projecting points of the stone would be subjected.

Common Lime Mortar. The first step in the preparation of common lime mortar is the slaking of the lime. This should be done by putting the lime into a water-tight box, or at least on a platform which is substantially water-tight, and on which a sort of pond is formed by a ring of sand. The amount of water to be used should be from $2\frac{1}{2}$ to 3 times the volume of the unslaked lime.

The "volume" of unslaked lime is a very uncertain quantity, varying with the amount of settlement caused by mere shaking which it may receive during transit. A *barrel* of lime means 230 pounds. If the barrel has a volume of 3.75 cubic feet, it would be just filled by 230 pounds of lime when this lime weighed about 61 pounds per cubic foot. This same lime, however, *may* be so shaken that it will weigh 75 pounds per cubic foot, in which case its volume is reduced to 81 per cent, or 3.05 cubic feet. Combining this with $2\frac{1}{2}$ to 3 times its volume of water will require about $8\frac{1}{2}$ cubic feet of water to one barrel of lime. On the other hand, if the lime has absorbed moisture

volume may become very materially increased.

Although close accuracy is not necessary, the lime paste will be injured if the amount of water is too much or too little. In short, the amount of water should be as near as possible that which is chemically required to hydrate the lime, so that on the one hand it shall be completely hydrated, and on the other hand it shall not be drowned in an excess of water which will injure its action in ultimate hardening. About three volumes of sand should be used to one volume of lime paste. Owing to the fact that the paste will, to a considerable extent, nearly fill the voids in the sand, the volume obtained from one barrel of unslaked lime made up into a mortar consisting of one part of lime paste to three parts of sand, will make about 6.75 barrels of mortar, or a little less than one cubic yard.

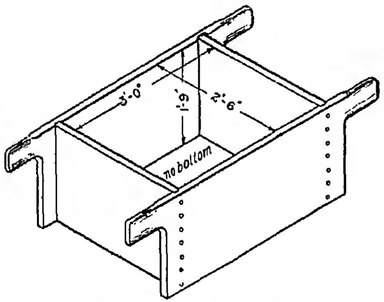


Fig. 9. Bottomless Box for Measuring Sand

Natural Cement Mortar. This is used, especially when mixed with lime to retard the setting, in the construction of walls of buildings, cellar foundations, and, in general, in masonry where the unit stresses are so low that strength is a minor consideration, but where a lime mortar would not harden because it is to be under water, or in a solid mass where the carbonic acid of the atmosphere could not penetrate to the interior. When natural cement is dumped loosely in a pile, the apparent volume is increased one-third or even one-half. This must be allowed for in mixing. A *barrel* averages 3.3 cubic feet. Therefore a 1:4 mortar of natural cement would require one barrel of cement to 13.2 cubic feet (about one-half a cubic yard) of sand. A bottomless box similar to that illustrated in Fig. 9, and with inside dimensions of 3 feet × 2 feet 6 inches × 1 foot 9 inches, contains 13.2 cubic feet. It is preferable to use even charges of one barrel of cement in mixing up a batch of mortar, rather than to dump it out and measure it loosely. If the size of the barrel varies from the average value given above, the size of the sand box should be varied accordingly. The barrels coming from any one cement

LEAMINGTON HOTEL, MINNEAPOLIS, MINNESOTA

This apartment hotel is one of the largest of its kind in the world, containing 360 suites of various sizes. The building is fireproof, of tile and concrete joist construction.

practically somewhat difficult to measure accurately the volume of a barrel, owing to its swelling form, it is best to fill a sample barrel with loose dry sand, and then to measure the volume of that sand by emptying it into a rectangular box whose inside area, together with the height of sand in it, can be readily measured.

Portland Cement Mortar. A barrel of Portland cement will contain 370 to 380 pounds, net, of cement. Its capacity averages about 3.3 cubic feet, although with some brands the capacity may reach 3.75. The expansion, when the cement is thrown loosely in a pile or into a measuring box, varies from 10 to 40 per cent. The subject will be discussed further under the head of "Concrete".

Lime in Cement Mortar. Lime is frequently employed in the cement mortar used for buildings, for a combination of reasons:

(*a*) It is unquestionably more economical; but if the percentage added (or that which replaces the cement) is more than about 5 per cent, the strength of the mortar is sacrificed. The percentage of loss of strength depends on the richness of the mortar.

(*b*) When used with a mortar leaner than 1:2, the substitution of about 10 per cent of lime for an equal weight of cement will render concrete more water-tight, although at some sacrifice in strength.

(*c*) It always makes the mortar *work* more easily and smoothly. In fact, a rich cement mortar is very *brash*; it will not stick to the bricks or stones when striking a joint. It actually increases the output of the masons to use a mortar which is rendered smoother by the addition of lime.

The substitution of more than 20 per cent of lime decreases the strength faster than the decrease in cost and therefore should not be permitted unless strength is a secondary consideration and the combination is considered more as an addition of cement to a lime mortar in order to render it hydraulic.

Effect of Re-Gaging or Re-Mixing Mortar. Specifications and textbooks have repeatedly copied from one another a requirement that all mortar which is not used immediately after being mixed and before it has taken an *initial set* must be rejected and thrown away. This specification is evidently based on the idea that after the initial set has been disturbed and destroyed, the cement no longer has the power of hardening, or at least that such power is very materially and seriously reduced. Repeated experiments, however, have shown that under some conditions the ultimate strength of the mortar (or concrete) is actually increased, and that it is not seriously injured

even when the mortar is re-gaged several hours after being originally mixed with water.

Effect on Lime Paste. Such a specification against re-mixing is never applied to lime paste, since it is well known that a lime paste is considerably improved by being left for several days (or even months) before being used. This is evidently due to the fact that even during such a period the carbonic acid of the atmosphere cannot penetrate appreciably into the mass of the paste, while the greater length of time merely insures a more perfect slaking of the lime. The presence of free unslaked lime in either lime or cement mortar is always injurious, because it generally results in expansion and disruption and possibly in injurious chemical reaction.

Effect on Portland Cement. Tests with Portland cement have shown that if it is re-mixed two hours after being combined with water, its strength, both tensile and compressive, is greater after six months' hardening, although it will be less after seven days' hardening, than in similar specimens which are molded immediately after mixing. It is also found that the re-mixing makes the cement much slower in its setting. The adhesion, moreover, is reduced by re-mixing, which is an important consideration in the use of reinforced concrete.

Effect on Natural Cement. The effects of tests with natural cement are somewhat contradictory, and this is perhaps the reason for the original writing of such a specification. The result of an elaborate series of tests made by Mr. Thomas F. Richardson showed that quick-setting cements which had been re-mixed showed a considerable falling off in strength in specimens broken after 7 days and 28 days of hardening, yet the ultimate strength after six months of hardening was invariably increased. It is also found that for both Portland and natural cements there is a very considerable increase in the strength of the mortar when it is worked continuously for two hours before molding or placing in the masonry. Such an increase is probably due to the more perfect mixing of the constituents of the mortar.

Conclusions. The conclusion of the whole matter appears to be that, when it is desirable that considerable strength shall be attained within a few days or weeks (as is generally the case, and especially so with reinforced-concrete work), the specification against re-mixing

should be rigidly enforced. For the comparatively few cases where a slow acquirement of the ultimate strength is permissible, re-mixing might be tolerated, although there is still the question whether the expected gain in ultimate strength would pay for the extra work. It would be seldom, if ever, that this claimed property of cement mortar could be relied on to save a batch of mortar which would otherwise be rejected because it had been allowed to stand after being mixed until it had taken an initial set.

Proportions of Materials for Mortar. *Lime Mortar.* As previously stated, p. 47, a barrel of unslaked lime should be mixed with about $8\frac{1}{2}$ cubic feet of water. This will make about 9 cubic feet of lime paste. Mixing this with a cubic yard of sand will make about 1 cubic yard of 1:3 lime mortar. This means approximately 1 volume of unslaked lime to 8 volumes of sand.

Cement Mortars. The volume of cement depends very largely on whether it is loosely dropped in a pile, shaken together, or packed. The practical commercial methods of obtaining a mixture of definite proportions will be given in the following section. Natural cement mortars are usually mixed in the 1:2 ratio, although a 1:1 mixture would be a safer mixture to use. Portland cement will be used to make 1:3 mortar for ordinary work, and 1:2 mortar for very high-grade work. As previously stated, a small percentage of lime is sometimes substituted for an equal volume of cement in order to make the mortar work better.

CONCRETE

CHARACTERISTICS AND PROPERTIES

Concrete is composed of a mixture of cement, sand, and crushed stone or gravel, which, after being mixed with water, soon sets and obtains a hardness and strength equal to that of a good building stone. These properties, together with its adaptability to monolithic construction, combined with its cheapness, render concrete very useful as a building material.

Principles Used in Proportioning Concrete. Theoretically, the proportioning of the sand and cementing material should be done by weight. It is always done in this way in laboratory testing. The volume of a given weight of cement is quite variable according as it is packed or loosely thrown in a pile. The same statement is

true of sand. A barrel of Portland cement will increase in volume from 10 to 30 per cent by being merely dumped loosely in a pile and then shoveled into a measuring box. In measuring the materials for concrete the cement should be measured in the original packages as it comes from the manufacturer, but the sand and stone should be measured loose as it is thrown in the measuring boxes. To a less extent uncertainty exists regarding the conditions of the sand. Loose dry sand occupies a considerably larger volume than wet sand, and this is still more the case when the sand is very fine.

Ideal Conditions. The general principle to be adopted is that the amount of water should be just sufficient to supply that needed for crystallization of the cement paste; that the amount of paste should be just sufficient to fill the voids between the particles of sand; that the mortar thus produced should be just sufficient to fill the voids between the broken stones. If this ideal could be realized, the total volume of the mixed concrete would be no greater than that of the broken stone. But no matter how thoroughly and carefully the ingredients are mixed and rammed, the particles of cement will get between the grains of sand and thus cause the volume of the mortar to be greater than that of the sand; the grains of sand will get between the smaller stones and separate them; and the smaller stones will get between the larger stones and separate them. Experiments by Prof. I. O. Baker have shown that, even when the volume of the mortar was only 70 per cent of the volume of the voids in the broken stone, the volume of the rammed concrete was 5 per cent more than that of the broken stone. When the theoretical amount of mortar was added, the volume was 7.5 per cent in excess, which shows that it is practically impossible to ram such concrete and wholly prevent voids. When mortar amounting to 110 per cent of the voids was used, all voids were apparently filled, but the volume of the concrete was 114 per cent of that of the broken stone.

Conditions in Practice. Therefore, on account of the impracticability of securing perfect mixing, the amount of water used is always somewhat in excess (which will do no harm); the cement paste is generally made somewhat in excess of that required to fill the particles in the sand (except in those cases where, for economy, the mortar is purposely made very lean); and the amount of mortar is usually considerably in excess of that required to fill the voids in

the stone. Even when we allow some excess in the above particulars, there is so much variation in the percentage of voids in the sand and broken stone, that the best work not only requires an experimental determination of the voids in the sand and stone which are being used; but, on account of the liability to variation in those percentages, even in materials from the same source of supply, the best work requires a constant testing and revision of the proportions as the work proceeds. For less careful work, the proportions ordinarily adopted in practice are considered sufficiently accurate.

Proportions. On the general principle that the voids in ordinary broken stone are somewhat less than half of the volume, it is a very common practice to use one-half as much sand as the volume of the broken stone. The proportion of cement is then varied according to the strength required in the structure, and according to the desire to economize. On this principle we have the familiar ratios 1:2:4, 1:2½:5, 1:3:6, and 1:4:8. It should be noted that in each of these cases, in which the numbers give the relative proportions of the cement, sand, and stone respectively, the ratio of the sand to the broken stone is a constant, and the ratio of the cement is alone variable, for it would be just as correct to express the ratios as follows: 1:2:4; 0.8:2:4; 0.67:2:4; 0.5:2:4.

Cinder Concrete. Cinder concrete has been used to some extent on account of its light weight. The strength of cinder concrete is from one-third to one-half the strength of stone concrete. It will weigh about 110 pounds per cubic foot.

Rubble Concrete. Rubble concrete is a concrete in which large stones are placed, and will be discussed in Part II.

Compressive Strength. The compressive strength of concrete is very important, as it is used more often in compression than in any other way. It is rather difficult to give average values of the compressive strength of concrete, as it is dependent on so many factors. The available aggregates are so varied, and the methods of mixing and manipulation so different, that tests must be studied before any conclusions can be drawn. For extensive work, tests should be made with the materials available to determine the strength of concrete, under conditions as nearly as possible like those in the actual structure.

A series of experiments made at the Watertown Arsenal for Mr. George A. Kimball, Chief Engineer of the Boston Elevated

been published, and the results are given in Table III. Portland cement, coarse sharp sand, and stone up to $2\frac{1}{2}$ inches were used; and when thoroughly rammed, the water barely flushed to the surface.

TABLE III

Compressive Strength of Concrete*

Tests Made at Watertown Arsenal, 1899

Mixture	Brand of Cement	Strength (Pounds per Square Inch)			
		7 Days	1 Month	3 Months	6 Months
1:2:4	Saylor	1724	2238	2702	3510
	Atlas	1387	2428	2966	3953
	Alpha	904	2420	3123	4411
	Germania	2219	2642	3082	3643
	Alsen	1592	2269	2608	3612
	Average	1565	2399	2896	3826
1:3:6	Saylor	1625	2568	2882	3567
	Atlas	1050	1816	1538	3170
	Alpha	892	2150	2355	2750
	Germania	1550	2174	2486	2930
	Alsen	1438	2114	2349	3026
	Average	1311	2164	2522	3088

The values obtained in these tests are exceedingly high, and cannot be safely counted on in practice.

Tests made by Prof. A. N. Talbot (University of Illinois, Bulletin No. 14) on 6-inch cubes of concrete, show the average values given in Table IV. The cubes were about 60 days old when tested.

TABLE IV

Compressive Tests of Concrete

University of Illinois

No. of Tests	Mixture	Strength (Pounds per Square Inch)
3	1:2:4	2350
6	1:3:5½	1920
7	1:3:6	1300

With fair conditions as to the character of the materials and workmanship, a mixture of 1:2:4 concrete should show a compressive

*From "Tests of Metals", 1899.

strength of 2000 to 2300 pounds per square inch in 40 to 60 days; a mixture of $1:2\frac{1}{2}:5$ concrete, a strength of 1800 to 2000 pounds per square inch; and a mixture of 1:3:6 concrete, a strength of 1500 to 1800 pounds per square inch. The rate of hardening depends upon the consistency and the temperature.

Tensile Strength. The tensile strength of concrete is usually considered about one-tenth of the compressive strength; that is, concrete which has a compressive value of 2000 pounds per square inch should have a tensile strength of about 200 pounds per square inch. Although there is no fixed relation between the two values, the general law of increase in strength due to increasing the percentage of cement and the density seems to hold in both cases.

Shearing Strength. The shearing strength of concrete is important on account of its intimate relation to the compressive strength and the shearing stresses to which it is subjected in structures reinforced with steel. But few tests have been made, as they are rather difficult to make; but the tests made show that the shearing strength of concrete is nearly one-half the crushing strength. By shearing is meant the strength of the material against a sliding failure when tested as a rivet would be tested for shear.

Modulus of Elasticity. The principal use of the modulus of elasticity in designing reinforced concrete is in determining the relative stresses carried by the concrete and the steel. The minimum value used in designing reinforced concrete is usually taken as 2,000,000, and the maximum value as 3,000,000, depending on the richness of the mixture used. A value of 2,500,000 is generally taken for ordinary concrete.

Weight. The weight of stone or gravel concrete will vary from 145 pounds per cubic foot to 155 pounds per cubic foot, depending upon the specific gravity of the materials and the degree of compactness. The weight of a cubic foot of concrete is usually considered as 150 pounds.

Cost. The cost of concrete depends upon the character of the work to be done and the conditions under which it is necessary to do this work. The cost of the material, of course, will always have to be considered, but this is not so important as the character of the work. The cost of concrete in place will range from $4.50 per cubic yard to $20, or even $25, per cubic yard. When it is laid in large

masses, so that the cost of forms is relatively small, the cost will range from \$4.50 per cubic yard to \$6 or \$7 per cubic yard, depending upon the local conditions and cost of materials. Foundations and heavy walls are good examples of this class of work. For sewers and arches, the cost will vary from \$7 to \$13. In building construction floors, roofs, and thin walls—the cost will range from \$14 to \$20 per cubic yard.

Cement. The cost of Portland cement varies with the demand. Being heavy, the freight is often a big item. The price varies from \$1 to \$2 per barrel. To this must be added the cost of handling.

Sand. The cost of sand, including handling and freight, ranges from \$0.75 to \$1.50 per cubic yard. A common price for sand delivered in the cities is \$1.00 per cubic yard.

Broken Stone or Gravel. The cost of broken stone delivered in the cities varies from \$1.25 to \$1.75 per cubic yard. The cost of gravel is usually a little less than stone.

Mixing. Under ordinary conditions and where the concrete will have to be wheeled only a very short distance, the cost of hand-mixing and placing will generally range from \$0.90 to \$1.30 per cubic yard, if done by men skilled in this work. If a mixer is used, the cost will range from \$0.50 to \$0.90 per cubic yard.

Forms. The cost of forms for heavy walls and foundations, varies from \$0.70 to \$1.20 per cubic yard of concrete laid. The cost of forms and mixing concrete will be further discussed in Part IV.

MIXING AND LAYING CONCRETE

Practical Methods of Proportioning. *Rich Mixture.* A rich mixture, proportions 1:2:4—that is, 1 barrel (4 bags) packed Portland cement (as it comes from the manufacturer), 2 barrels (7.6 cubic feet) loose sand, and 4 barrels (15.2 cubic feet) loose stone is used in arches, reinforced-concrete floors, beams, and columns for heavy loads; engine and machine foundations subject to vibration; tanks; and for water-tight work.

Medium Mixture. A medium mixture, proportions 1:2½:5 that is, 1 barrel (4 bags) packed Portland cement, 2½ barrels (9.5 cubic feet) loose sand, and 5 barrels (19 cubic feet) loose gravel or stone—may be used in arches, thin walls, floors, beams, sewers, sidewalks, foundations, and machine foundations.

TABLE V

Proportions of Cement, Sand, and Stone in Actual Structures

Structure	Proportions	Reference
C. B. & Q. R. R.		
Reinforced Concrete Culverts....	1:3:6	Engr. Cont. Oct. 3, '06
Phila. Rapid Transit Co.		
Floor Elevated Roadway........	1:3:6	" " Sept. 26, '06
Subway { Walls.	1:2.5:5	
Subway { Floors...............	1:3:6	
C. P. R. R.		
Arch Rings....................	1:3:5	
Piers and Abutments...........	1:4:7	Cement Era, Aug. '06
Hudson River Tunnel Caisson.....	1:2:4	Eng. Record, Sept. 29, '06
Stand Pipe at Attleboro, Mass. .... Height, 106 feet.	1:2:4	" " " 29, '06
C.C. & St.L.R.R., Danville Arch		
Footings......................	1:4:8 or 1:9.5	" " March 3, '06
Arch Rings....................	1:2:4	
Abutments, Piers...............	1:3:6 or 1:6.5	
N. Y. C. & H. R. R. R.		
Ossining Tunnel { Footing.............	1:4:7.5	" " " 3, '06
Ossining Tunnel { Walls...............	1:3:6	
Ossining Tunnel { Coping..............	1:2:4	
American Oak Leather Co.		
Factory at Cincinnati, Ohio.	1:2:4	" " " 3, '06
Harvard University Stadium......	1:3:6	
New York Subway		
Roofs and Sidewalks...........	1:2:4	
Tunnel Arches.................	1:2.5:5	
Wet Foundation 2′ th. or less....	1:2:4	
Wet Foundation exceeding 2′....	1:2.5:5	
Boston Subway..................	1:2.5:4	
P. & R. R. R.		
Arches........................	1:2:4	" " Oct. 13, '06
Piers and Abutments...........	1:3:6	
Brooklyn Navy Yd. Laboratory		
Columns.......................	1:2:3 Trap rock	Eng. News, March 23, '05
Beams and Slabs...............	1:3:5 Trap rock	
Roof Slab.....................	1:3:5 Cinder	
Southern Railway.		
Arches........................	1:2:4	
Piers and Abutments...........	1:2.5:5	

Ordinary Mixture. An ordinary mixture, proportions 1:3:6—that is, 1 barrel (4 bags) packed Portland cement, 3 barrels (11.4 cubic feet) loose sand, and 6 barrels (22.8 cubic feet) loose gravel or broken stone—may be used for retaining walls, abutments, piers, and machine foundations.

Lean Mixture. A lean mixture, proportions 1:4:8—that is, 1 barrel (4 bags) packed Portland cement, 4 barrels (15.2 cubic feet) loose sand, and 8 barrels (30.4 cubic feet) loose gravel or broken stone—may be used in large foundations supporting stationary loads, backing for stone masonry, or where it is subject to a low compressive load.

Tendency Towards Richer Mixtures. These proportions must not be taken as always being the most economical to use, but they represent average practice. Cement is the most expensive ingredient; therefore a reduction of the quantity of cement, by adjusting the proportions of the aggregate so as to produce a concrete with the same density, strength, and impermeability, is of great importance. By careful proportioning and workmanship, water-tight concrete has been made of a 1:3:6 mixture.

In the last few years the tendency throughout the country has been to use a richer mixture than was formerly used for reinforced concrete. The 1:2:4 mixture is now used practically for all buildings constructed of reinforced concrete, even if low stresses are used, but theoretically a 1:2½:5 mixture should have sufficient strength.

In Table V will be found the proportions of the concrete used in various well-known structures and in Tables VI to IX the amounts of materials used per cubic yard for various proportions.

Proper Proportions Determined by Trial. An accurate and simple method to determine the proportions of concrete is by trial batches. The apparatus consists of a scale and a cylinder which may be a piece of wrought-iron pipe from 10 to 12 inches in diameter capped at one end. Measure and weigh the cement, sand, stone, and water and mix on a piece of sheet steel, the mixture having a consistency the same as to be used in the work. The mixture is placed in the cylinder, carefully tamped, and the height to which the pipe is filled is noted. The pipe should be weighed before and after being filled so as to check the weight of the material. The

of Cement Paste.)

Proportion of Cement to Sand	1:1	1:1.5	1:2	1:2.5	1:3	1:4
	Bbls.	Bbls.	Bbls.	Bbls.	Bbls.	Bbls.
Bbl. specified to be 3.5 cu. ft.	4.22	3.49	2.97	2.57	2.28	1.76
" " " 3.8 "	4.09	3.33	2.81	2.45	2.16	1.62
" " " 4.0 "	4.00	3.24	2.73	2.36	2.08	1.54
" " " 4.4 "	3.81	3.07	2.57	2.27	2.00	1.40
Cu. yds. sand per cu. yd. mortar. .	0.6	0.7	0.8	0.9	1.0	1.0

of the sand and stone but having the same total weight as before. Note the height in the cylinder, which will be a guide to other batches to be tried. Several trials are made until a mixture is found that gives the least height in the cylinder, and at the same time works well while mixing, all the stones being covered with mortar, and which makes a good appearance. This method gives very good results, but it does not indicate the changes in the physical sizes of the sand and stone so as to secure the most economical composition, as would be shown in a thorough mechanical analysis.

There has been much concrete work done where the proportions were selected without any reference to voids, which has given much better results in practice than might be expected. The proportion of cement to the aggregate depends upon the nature of the construction and the required degree of strength, or water-tightness, as well as upon the character of the inert materials. Both strength and imperviousness increase with the proportion of cement to the aggregate. Richer mixtures are necessary for loaded columns, beams in building construction and arches for thin walls subject to water pressure, and for foundations laid under water. The actual measurements of materials as actually mixed and used usually show leaner mixtures than the nominal proportions specified. This is largely due to the heaping of the measuring boxes.

Methods of Mixing. The method of mixing concrete is immaterial, if a homogeneous mass, containing the cement, sand, and stone in the correct proportions is secured. The value of the concrete depends greatly upon the thoroughness of the mixing. The color of the mass must be uniform and every grain of sand and piece of the stone should have cement adhering to every point of its surface.

TABLE VII

Barrels of Portland Cement Per Cubic Yard of Mortar

(Voids in Sand Being 45 per cent and 1 Bbl. Cement Yielding 3.4 Cu
of Cement Paste.)

Proportions of Cement to Sand	1:1	1:1.5	1:2	1:2.5	1:3
	Bbls.	Bbls.	Bbls.	Bbls.	Bbls
Bbl. specified to be 3.5 cu. ft.	4.62	3.80	3.25	2.84	2.3
" " " 3.8 "	4.32	3.61	3.10	2.72	2.1
" " " 4.0 "	4.19	3.46	3.00	2.64	2.0
" " " 4.4 "	3.94	3.34	2.90	2.57	1.8
Cu. yds. sand per cu. yds. mortar.	0.6	0.8	0.9	1.0	1.0

TABLE VIII

Ingredients in 1 Cubic Yard of Concrete

(Sand Voids, 40 per cent, Stone Voids, 45 per cent; Portland Cemen
Yielding 3.65 Cubic Feet Paste. Barrel Specified to be 3.8 Cubic

Proportions by Volume	1:2:4	1:2:5	1:2:6	1:2.5:5	1:2.5
Bbls. cement per cu. yd concrete.	1.46	1.30	1.18	1.13	1.0
Cu. yds. sand " " .	0.41	0.36	0.33	0.40	0.3
" stone " " .	0.82	0.90	1.00	0.80	0.8
Proportions by volume.	1:3:5	1:3:6	1:3:7	1:4:7	1:4:
Bbls. cement per cu. yd. concrete.	1.13	1.05	0.96	0.82	0.7
Cu. yds. sand " " .	0.48	0.44	0.40	0.46	0.4
" stone " " .	0.80	0.88	0.93	0.80	0.8

This table is to be used when cement is measured packed in th
for the ordinary barrel holds 3.8 cubic feet.

TABLE IX

Ingredients in 1 Cubic Yard of Concrete

(Sand Voids, 40 per cent; Stone Voids, 45 per cent; Portland Ceme
Yielding 3.65 Cubic Feet of Paste. Barrel Specified to be 4.4 Cu

Proportions by Volume	1:2:4	1:2:5	1:2:6	1:2.5:5	1:2.5
Bbls. cement per cu. yd. concrete.	1.30	1.16	1.00	1.07	0.9
Cu. yds. sand " " .	0.42	0.38	0.33	0.44	0.4
" stone " " .	0.84	0.95	1.00	0.88	0.9
Proportions by volume.	1:3:5	1:3:6	1:3:7	1:4:7	1:4:
Bbls. cement per cu. yd. concrete.	0.96	0.90	0.82	0.75	0.6
Cu. yds. sand " " .	0.47	0.44	0.40	0.49	0.4

Wetness of Concrete. In regard to plasticity, or facility c working and molding, concrete may be divided into three classes dry, medium, and very wet.

Dry Concrete. Dry concrete is used in foundations which ma be subjected to severe compression a few weeks after being placec It should not be placed in layers of more than 8 inches, and shoul be thoroughly rammed. In a dry mixture the water will just flus to the surface only when it is thoroughly tamped. A dry mixtur sets and will support a load much sooner than if a wetter mixtur is used, and generally is used only where the load is to be applie soon after the concrete is placed. This mixture requires the exercis of more than ordinary care in ramming, as pockets are apt to b formed in the concrete; and one argument against it is the difficult of getting a uniform product.

Medium Concrete. Medium concrete will quake when rammec and has the consistency of liver or jelly. It is adapted for construc tion work suited to the employment of mass concrete, such as retair ing walls, piers, foundations, arches, abutments; and is sometime also employed for reinforced concrete.

Very Wet Concrete. A very wet mixture of concrete will ru off a shovel unless it is handled very quickly. An ordinary ramme will sink into it of its own weight. It is suitable for reinforced cor crete, such as thin walls, floors, columns, tanks, and conduits.

Modern Practice. Within the last few years there has been marked change in the amount of water used in mixing concret The dry mixture has been superseded by a medium or very we mixture, often so wet as to require no ramming whatever. Exper ments have shown that *dry mixtures* give better results in *short tin tests* and *wet mixtures* in *long time tests.* In some experiments mac on dry, medium, and wet mixtures it was found that the mediur mixture was the most dense, wet next, and dry least. This exper menter concluded that the medium mixture is the most desirabl since it will not quake in handling but will quake under heav

some engineers think that the tendency is to use far too much rather than too little water, but that thorough ramming is desirable. In thin walls very wet concrete can be more easily pushed from the surface so that the mortar can get against the forms and give a smooth surface. It has also been found essential that the concrete should be wet enough so as to flow under and around the steel reinforcement so as to secure a good bond between the steel and concrete.

Following are the specifications (1903) of the American Railway Engineering and Maintenance of Way Association:

> "The concrete shall be of such consistency that when dumped in place it will not require tamping; it shall be spaded down and tamped sufficiently to level off and will then quake freely like jelly, and be wet enough on top to require the use of rubber boots by workman."

Transporting and Depositing Concrete. Concrete is usually deposited in layers of 6 inches to 12 inches in thickness. In handling and transporting concrete, care must be taken to prevent the separation of the stone from the mortar. The usual method of transporting concrete is by wheelbarrows, although it is often handled by cars and carts, and on small jobs it is sometimes carried in buckets. A very common practice is to dump it from a height of several feet into a trench. Many engineers object to this process as they claim that the heavy and light portions separate while falling and the concrete is therefore not uniform through its mass, and they insist that it must be gently slid into place. A wet mixture is much easier to handle than a dry mixture, as the stone will not so readily separate from the mass. A very wet mixture has been deposited from the top of forms 43 feet high and the structure was found to be waterproof. On the other hand, the stones in a dry mixture will separate from the mortar on the slightest provocation. Where it is necessary to drop a dry mixture several feet, it should be done by means of a chute or pipe.

Ramming Concrete. Immediately after concrete is placed, it should be rammed or puddled, care being taken to force out the air bubbles. The amount of ramming necessary depends upon how much water is used in mixing the concrete.

A rammer for dry concrete is shown in Fig. 10; and one for wet concrete, in Fig. 11. In very thin walls, where a wet mixture is used, often the tamping or puddling is done with a part of a reinforcing bar. A common spade is often employed for the face of work, being used to push back stones that may have separated from the mass, and also to work the finer portions of the mass to the face, the method being to work the spade up and down the face until it is thoroughly filled. Care must be taken not to pry with the spade, as this will spring the forms unless they are very strong.

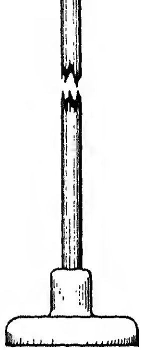


Fig. 10. Rammer for Dry Concrete

Bonding Old and New Concrete. To secure a water-tight joint between old and new concrete requires a great deal of care. Where the strain is chiefly compressive, as in foundations, the surface of the concrete laid on the previous day should be washed with clean water, no other precautions being necessary. In walls and floors, or where a tensile stress is apt to be applied, the joint should be thoroughly washed and soaked, and then painted with neat cement or a mixture of one part cement and one part sand, made into a very thin mortar.

In the construction of tanks or any other work that is to be water-tight, in which the concrete is not placed in one continuous operation, one or more square or V-shaped joints are necessary. These joints are formed by a piece of timber, say 4 inches by 6 inches, being imbedded in the surface of the last concrete laid each day. On the following morning, when the timber is removed, the joint is washed and coated with neat cement or 1:1 mortar. The joints may be either horizontal or vertical. The bond between old and new concrete may be aided by roughening the surface after ramming or before placing the new concrete.

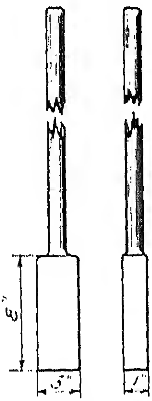


Fig. 11. Rammer for Wet Concrete

now generally accepted that the ultimate effect of freezing of Portland cement concrete is to produce only a surface injury. The setting and hardening of Portland cement concrete is retarded, and the strength at short periods is lowered, by freezing; but the ultimate strength appears to be only slightly, if at all, affected. A thin scale about $\frac{1}{16}$ inch in depth is apt to scale off from granolithic or concrete pavements which have been frozen, leaving a rough instead of a troweled wearing surface; and the effect upon concrete walls is often similar; but there appears to be no other injury. Concrete should not be laid in freezing weather, if it can be avoided, as this involves additional expense and requires greater precautions to be taken; but with proper care, Portland cement concrete can be laid at almost any temperature.

Preventive Methods. There are three methods which may be used to prevent injury to concrete when laid in freezing weather:

First: Heat the sand and stone, or use hot water in mixing the concrete.

Second: Add salt, calcium chloride, or other chemicals to lower the freezing point of the water.

Third: Protect the green concrete by enclosing it and keeping the temperature of the enclosure above the freezing point.

The first method is perhaps more generally used than either of the others. In heating the aggregate, the frost is driven from it; hot water alone is insufficient to get the frost out of the frozen lumps of sand. If the heated aggregate is mixed with water which is hot but not boiling, experience has shown that a comparatively high temperature can be maintained for several hours, which will usually carry it through the initial set safely. The heating of the materials also hastens the setting of the cement. If the fresh concrete is covered with canvas or other material, it will assist in maintaining a higher temperature. The canvas, however, must not be laid directly on the concrete, but an air space of several inches must be left between the concrete and the canvas.

The aggregate is heated by means of steam pipes laid in the bottom of the bins, or by having pipes of strong sheet iron, about

18 inches in diameter, laid through the bottom of the bins, and fires built in the pipes. The water may be heated by steam jets or other means. It is also well to keep the mixer warm in severe weather, by the use of a steam coil on the outside, and jets of steam on the inside.

The second method—lowering the freezing point by adding salt—has been commonly used to lower the freezing point of water. Salt will increase the time of setting and lower the strength of the concrete for short periods. There is a wide difference of opinion as to the amount of salt that may be used without lowering the ultimate strength of the concrete. Specifications for the New York Subway work required nine pounds of salt to each 100 pounds (12 gallons) of water in freezing weather. A common rule calls for 10 per cent of salt to the weight of water, which is equivalent to about 13 pounds of salt to a barrel of cement.

The third method is the most expensive, and is used only in building construction. It consists in constructing a light wood frame over the site of the work, and covering the frame with canvas or other material. The temperature of the enclosure is maintained above the freezing point by means of stoves.

WATERPROOFING CONCRETE

Concrete Not Generally Water-Tight. Concrete as ordinarily mixed and placed is not generally water-tight, but experience has shown that where concrete is proportioned to obtain the greatest practicable density and is mixed wet the resulting concrete is impervious under a moderate pressure. With the wet mixtures of concrete now generally used in engineering work, concrete possesses far greater density, and is correspondingly less porous, than with the older, dryer mixtures. However, it is difficult, on large masses of actual work, to produce concrete of such close texture as to prevent seepage at all points. It has frequently been observed that when concrete was green there was a considerable seepage through it, and that in a short time all seepage stopped. Concrete has been made practically water-tight by forcing through it water which contained

surface to make the concrete water-tight. Many of the compounds are of but temporary value and in time lose their usefulness as a waterproofing material.

Effect of Steel Reinforcement. Reinforcing steel properly proportioned and located both horizontally and vertically in long walls, subways, and reservoirs, will greatly assist in rendering the concrete impervious by reducing the cracks so that if they do occur they will be too minute to permit leakage, or the small cracks will soon fill up with silt.

Coatings Applied on Pressure Side of Walls. Several successful methods of waterproofing will be given here, and most of these methods will also apply to stone and brickwork. In the operation of waterproofing, a very common mistake is made in applying the waterproofing materials on the wrong side of the wall to be made water-tight. That is, if water finds its way through a cellar wall, it is useless to apply a waterproofing coat on the inside surface of the wall, as the pressure of the water will push it off. If, however, there is no great pressure behind it, a waterproofing coat applied on the inside of the wall may be successful in keeping moisture out of the cellar. To be successful in waterproofing a cellar wall, the waterproofing material should be applied on the *outside* surface of the wall; and if properly applied, the wall, as well as the cellar, will be entirely free of water.

In tank or reservoir construction, the conditions are different, in that it is generally desired to prevent the escape of water. In these cases, therefore, the waterproofing is applied on the inside surface, and is supported by the materials used in constructing the tank or reservoir. The structure should always be designed so that it can be properly waterproofed, and the waterproofing should always be applied on the side of the wall on which the pressure exists.

Waterproofing Methods. *Plastering.* For cisterns, swimming pools, or reservoirs, two coats of Portland cement grout—1 part cement, 2 parts sand—applied on the inside, have been used to make the concrete water-tight. One inch of rich mortar has usually been found effective under medium pressure.

At Attleboro, Mass., a large reinforced concrete standpipe, 50 feet in diameter, 106 feet high from the inside of the bottom to the

The walls of the standpipe are 18 inches thick at the bottom and 8 inches thick at the top. A mixture of 1 part cement, 2 parts sand, and 4 part broken stone, the stone varying from $\frac{1}{4}$ inch to $1\frac{1}{2}$ inches, was used. The forms were constructed, and the concrete placed, in sections of 7 feet. When the walls of the tank had been completed, there was some leakage at the bottom with a head of water of 100 feet. The inside walls were then thoroughly cleaned and picked, and four coats of plaster applied. The first coat contained 2 per cent of lime to 1 part of cement and 1 part of sand; the remaining three coats were composed of 1 part sand to 1 part cement. Each coat was floated until a hard dense surface was produced; then it was scratched to receive the succeeding coat.

On filling the standpipe after the four coats of plaster had been applied, the standpipe was found to be not absolutely water-tight. The water was drawn out; and four coats of a solution of castile soap, and one of alum, were applied alternately; and, under a 100-foot head, only a few leaks then appeared. Practically no leakage occurred at the joints; but in several instances a mixture somewhat wetter than usual was used, with the result that the spading and ramming served to drive the stone to the bottom of the batch being placed, and, as a consequence, in these places porous spots occurred. The joints were obtained by inserting beveled tonguing pieces, and by thoroughly washing the joint and covering it with a layer of thin grout before placing additional concrete.

Alum and Soap; Linseed Oil. Mortar may be made practically non-absorbent by the addition of alum and potash soap. One per cent by weight of powdered alum is added to the dry cement and sand, and thoroughly mixed; and about one per cent of any potash soap (ordinary soft soap) is dissolved in the water used in the mortar. A solution consisting of 1 pound of concentrated lye, 5 pounds of alum, and 2 gallons of water, applied while the concrete is green and until it lathers freely, has been successfully used. Coating the surface with boiled linseed oil until the oil ceases to be absorbed is another method that has been used with success.

Hydrated Lime. Hydrated lime has been successfully used to render concrete impervious. The very fine particles of the lime fill

the voids that would be otherwise left, thereby, increasing the density of the concrete. For a 1:2:4 concrete hydrated lime amounting to six to eight per cent of the weight of the cement is used. When it is used in a leaner mixture the percentage of lime is increased, that is, for a 1:3:6 concrete a percentage of lime up to 16 or 18 per cent is sometimes used.

Sylvester Process. The alternate application of washes of castile soap and alum, each being dissolved in water, is known as the *Sylvester process* of waterproofing. Castile soap is dissolved in water, $\frac{3}{4}$ of a pound of soap to a gallon of water, and applied boiling hot to the concrete surface with a flat brush, care being taken not to form a froth. The alum dissolved in water—1 pound pure alum in 8 gallons of water—is applied 24 hours later, the soap having had time to become dry and hard. The second wash is applied in the same manner as the first, at a temperature of 60 to 70 degrees Fahrenheit. The alternate coats of soap and alum are repeated every 24 hours. Usually four coats will make an impervious coating. The soap and alum combine and form an insoluble compound, filling the pores of the concrete and preventing the seepage of water. The walls should be clean and dry, and the temperature of the air not lower than 50 degrees Fahrenheit, when the composition is applied. The composition should be applied while the concrete is still green. This method of waterproofing has been used extensively for years, and has generally given satisfactory results for moderate pressures.

Asphalt. Asphalt is laid in thicknesses from $\frac{1}{4}$ inch to 1 inch as a waterproofing course. It is usually laid in one or more continuous sheets. It is also used for filling in contraction joints in concrete. The backs of retaining walls, of either concrete, stone, or brick, are often coated with asphalt to make them waterproof, the asphalt being applied hot with a mop. The bottoms of reservoirs have been constructed of concrete blocks six to eight feet square with asphalt joints $\frac{3}{8}$ inch to $\frac{1}{2}$ inch in thickness and extending at least halfway through the joint, that is, for a block 6 inches in thickness the asphalt would extend down at least 3 inches.

Asphalt is a mineral substance composed of different hydrocarbons, which are widely scattered throughout the world. There is a great variety of forms in which it is found, ranging from volatile liquids to thick semi-fluids and solids. These are usually inter-

mixed with different kinds of inorganic or organic matter, but are sometimes found in a free or pure state. Liquid varieties are known as *naphtha* and *petroleum;* the viscous or semi-fluid as *maltha* or *mineral tar;* and the solid as *asphalt* or *asphaltum.* The most noted deposit of asphalt is found in the island of Trinidad and at Bermudez, Venezuela, which is used extensively in this country for paving and roofing materials. The bituminous limestone deposited at Seyssel and Pyrimont, France; in Val-de-Travers, Canton of Neuchatel, Switzerland; and at Ragusa, Sicily are known as rock asphalt and are perhaps the best for waterproofing purposes.

In the construction of the filter plant at Lancaster, Pa., in 1905, a pure-water basin and several circular tanks were constructed of reinforced concrete. The pure-water basin is 100 feet wide by 200 feet long and 14 feet deep, with buttresses spaced 12 feet 6 inches center to center. The walls at the bottom are 15 inches thick, and 12 inches thick at the top. Four circular tanks are 50 feet in diameter and 10 feet high, and eight tanks are 10 feet in diameter and 10 feet high. The walls are 10 inches thick at the bottom, and 6 inches at the top. A wet mixture of 1 part cement, 3 parts sand, and 5 parts stone was used. No waterproofing material was used in the construction of the tanks; and when tested, two of the 50-foot tanks were found to be water-tight, and the other two had a few leaks where wires which had been used to hold the forms together had pulled out when the forms were taken down. These holes were stopped up and no further trouble was experienced. In constructing the floor of the pure-water basin, a thin layer of asphalt was used as shown in Fig. 12, but no waterproofing material was used in the walls, and both were found to be water-tight.

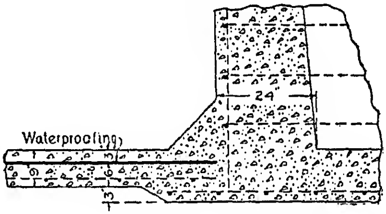


Fig. 12. Floor of Pure-Water Basin

Felt Laid with Asphalt. Alternate layers of paper or felt are laid with asphalt or tar, and are frequently used to waterproof floors, tunnels, subways, roofs, arches, etc. These materials range from ordinary tar paper laid with coal-tar pitch or asphalt to asbestos or asphalt felt laid in coal-tar or asphalt. Coal-tar products have

come into very common use for this work but the coal-tar should contain a large percentage of carbon to be satisfactory.

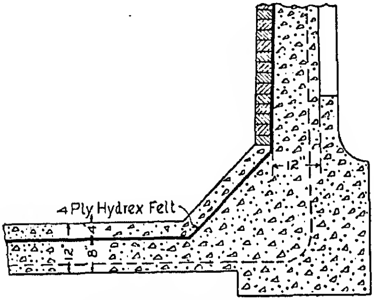


Fig. 13. Method of WaterProofing Reservoirs by Means of "Hydrex" Felt

In using these materials for rendering concrete water-tight, usually a layer of concrete or brick is first laid. On this is mopped a layer of hot asphalt; felt or paper is then laid on the asphalt, the latter being lapped from 6 to 12 inches. After the first layer of felt is placed, it is mopped over with hot asphalt compound, and another layer of felt or paper is laid, the operation being repeated until the desired thickness is secured, which is usually from 2 to 10 layers—or, in other words, the waterproofing varies from 2-ply to 10-ply. A waterproofing course of this kind, or a course as described in the paragraph on asphalt waterproofing, forms a distinct joint, and the

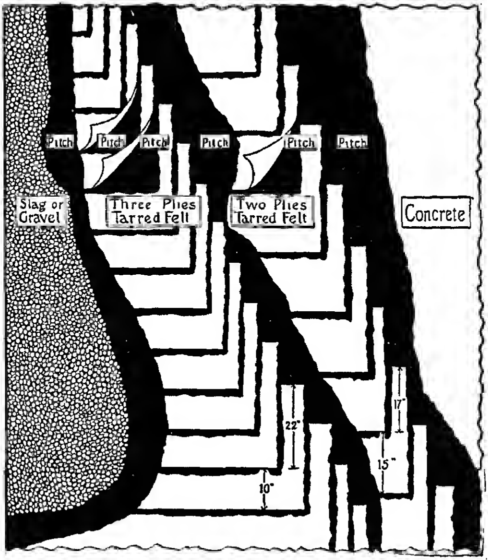


Fig. 14. Section Showing Method of Waterproofing Concrete
Courtesy of Barrett Manufacturing Company

When asphalt, or asphalt laid with felt paper, is used for waterproofing the interiors of the walls of tanks, a 4-inch course of brick is required to protect and hold in place the waterproofing materials. Fig. 13 shows a wall section of a reservoir (*Engineering Record*, Sept. 21, 1907) constructed for the New York, New Haven and Hartford Railroad, which illustrates the methods described above. The waterproofing materials for this reservoir consist of 4-ply "Hydrex" felt, and "Hydrex" compound was used to cement the layers together.

Fig. 14 is an illustration of the method used by the Barrett Manufacturing Company in applying their 5-ply coal-tar pitch and felt roofing material. It illustrates in a general way the method used in applying waterproofing. The surfaces to be waterproofed are mopped with pitch or asphalt. While the pitch is still hot, a layer of felt is placed, which is followed with alternate layers of pitch or asphalt until the required number of layers of felt has been secured. In no place should one layer of felt be permitted to touch the layer above or below it. When the last layer of felt is laid and thoroughly mopped with the coal-tar, something should be placed over the entire surface waterproofed to protect it from being injured. For roofing, this protection is gravel, as shown in Fig. 14. In waterproofing the back of concrete or stone arches usually a layer of brick is placed and then the joints between the bricks are filled with pitch. Brick used in this manner also assist in holding the waterproofing in place. Five layers of felt and pitch should be a sufficient protection against a head of water of ten feet.

PRESERVATION OF STEEL IN CONCRETE

Short Time Tests. Tests have been made to find the value of Portland cement concrete as a protection of steel or iron from corrosion. Nearly all of these tests have been of short duration (from a few weeks to several months); but they have clearly shown, when the steel or iron is properly imbedded in concrete, that on being removed therefrom it is clean and bright. Steel removed from concrete containing cracks or voids usually shows rust at the points where the voids or cracks occur; but if the steel has been *completely covered* with concrete, there is no corrosion. Tests have shown that if corroded steel is imbedded in concrete, the concrete will remove the rust. To secure the best results, the concrete should be mixed

quite wet, and care should be taken to have the steel thoroughly imbedded in the concrete.

Cinder vs. Stone Concrete. A compact cinder concrete has proven about as effective a protection for steel as stone concrete. The corrosion found in cinder concrete is mainly due to iron oxide or rust in the cinders, and not to the sulphur. The amount of sulphur in cinders is extremely small, and there seems to be little danger from that source. A steel-frame building erected in New York in 1898 had all its framework, except the columns, imbedded in cinder concrete; when the building was demolished in 1903, the frame showed practically no rust which could be considered as having developed after the material was imbedded.

Practical Illustrations. Cement washes, paints, and plasters have been used for a long time, in both the United States and Europe, for the purpose of protecting iron and steel from rust. The engineers of the Boston Subway, after making careful tests and investigations, adopted Portland cement paint for the protection of the steel work in that structure. The railroad companies of France use cement paint extensively to protect their metal bridges from corrosion. Two coats of the cement paint and sand are applied with leather brushes.

A concrete-steel water main on the Monier system, 12 inches in diameter, 1.6 inches thick, containing a steel framework of $\frac{1}{4}$-inch and $\frac{1}{16}$ inch steel rods, was taken up after 15 years' use in wet ground, at Grenoble, France. The adhesion was found perfect, and the metal absolutely free from rust.

William Sooy Smith, M. Am. Soc. C. E., states that in removing a bed of concrete at a lighthouse in the Straits of Mackinac, twenty years after it was laid, and ten feet below water surface, imbedded iron drift-bolts were found free from rust.

A very good example of the preservation of steel imbedded in concrete is given by Mr. H. C. Turner (*Engineering News*, January 16, 1908). Mr. Turner's company has recently torn down a one-story reinforced-concrete building erected by his company in 1902, at New Brighton, Staten Island. The building had a pile foundation, the piles being cut off at mean tide level. The footings, side walls, columns, and roof were all constructed of reinforced concrete. The portion removed was 30 by 60 feet, and was razed to make room for a five-story building. In concluding his account, Mr. Turner says:

few cases where the hoops were allowed to come closer than ¾ inch to the surface. Some evidence of corrosion was found in such cases, thus demonstrating the necessity of keeping the steel reinforcement at least ¾ inch from the surface. The footings were covered by the tide twice daily. The concrete was extremely hard, and showed no weakness whatever from the action of the salt water. The steel bars in the footings were perfectly preserved, even in cases where the concrete protection was only ¾ inch thick."

Tests by Professor Norton. Prof. Chas. L. Norton made several experiments with concrete bricks, 3 by 3 by 8 inches, in which steel rods, sheet metal, and expanded metal were imbedded. The specimens were enclosed in tin boxes with unprotected steel and were exposed for three weeks. One portion was exposed to steam, air, and carbon dioxide; another to air and steam; another to air and carbon dioxide; and another was left in the testing room. In these tests, Portland cement was used. The bricks were made of neat cement of 1 part cement and 3 parts sand; of 1 part cement and 5 parts stone; and of 1 part cement and 7 parts cinders. After the steel had been imbedded in these blocks three weeks, they were opened and the steel examined and compared with specimens which had been unprotected in corresponding boxes in the open air. The unprotected specimens consisted of rather more rust than steel; the specimens imbedded in neat cement were found to be perfectly protected; the rest of the specimens showed more or less corrosion. Professor Norton's conclusions were as follows:

1. Neat Portland cement is a very effective preventive against rusting.
2. Concrete, to be effective in preventing rust, should be dense and without voids or cracks. It should be mixed wet when applied to steel.
3. The corrosion found in cinder concrete is mainly due to iron oxide in the cinders, and not to sulphur.
4. Cinder concrete, if free from voids and well rammed when wet, is about as effective as stone concrete.
5. It is very important that the steel be clean when imbedded in concrete.

FIRE PROTECTIVE QUALITIES OF CONCRETE

High Resisting Qualities. The various tests which have been conducted—including the involuntary tests made as the result of fires—have shown that the fire-resisting qualities of concrete, and even resistance to a combination of fire and water, are greater than those of any other known type of building construction. Fires and experiments which test buildings of reinforced concrete have proved

Fahrenheit, the surface of the concrete may be injured to a depth of $\frac{1}{2}$ to $\frac{3}{4}$ inch or even of one inch; but the body of the concrete is not affected, and the only repairs required, if any, consist of a coat of plaster.

Thickness of Concrete Required for Fireproofing. Actual fires and tests have shown that 2 inches of concrete will protect an I-beam with good assurance of safety. Reinforced concrete beams and girders should have a clear thickness of $1\frac{1}{2}$ inches of concrete outside the steel on the sides and 2 inches on the bottom; slabs should have at least 1 inch below the slab bars, and columns 2 inches. Structural steel columns should have at least 2 inches of concrete outside of the farthest projecting edge.

Theory. The theory of the fireproofing qualities of Portland cement concrete given by Mr. Spencer B. Newberry is that the capacity of the concrete to resist fire and prevent its transference to steel is due to its *combined water and porosity*. In hardening, concrete takes up 12 to 18 per cent of the water contained in the cement. This water is chemically combined, and not given off at the boiling point. On heating, a part of the water is given off at 500 degrees Fahrenheit, but dehydration does not take place until 900 degrees Fahrenheit is reached. The mass is kept for a long time at comparatively low temperature by the vaporization of water absorbing heat. A steel beam imbedded in concrete is thus cooled by the volatilization of water in the surrounding concrete.

Resistance to the passage of heat is offered by the porosity of concrete. Air is a poor conductor, and an air space is an efficient protection against conduction. The outside of the concrete may reach a high temperature; but the heat only slowly and imperfectly penetrates the mass, and reaches the steel so gradually that it is carried off by the metal as fast as it is supplied.

Cinder vs. Stone Concrete. Mr. Newberry says: "Porous substances, such as asbestos, mineral wool, etc., are always used as heat-insulating material. For this same reason, cinder concrete, being highly porous, is a much better non-conductor than a dense concrete made of sand and gravel or stone, and has the added advantage of being light."

Professor Norton, on the other hand, in comparing the actions of

cinder and stone concrete in the great Baltimore fire of February, 1904, states that there is but little difference in the two concretes. The burning of bits of coal in poor cinder concrete is often balanced by the splitting of stones in the stone concrete. "However, owing to its density, the stone concrete takes longer to heat through."

Fire and Water Tests. Under the direction of Prof. Francis C. Van Dyck, a test was made on December 26, 1905, on stone and cinder reinforced concrete, according to the standard fire and water tests of the New York Building Department. A building was constructed 16 feet by 25 feet, with a wall through the middle. The roof consisted of the two floors to be tested. One floor was a reinforced cinder concrete slab and steel I-beam construction; and the other was a stone concrete slab and beam construction. The floors were designed for a safe load of 150 pounds per square foot, with a factor of safety of four.

The object of the test was to ascertain the result of applying to these floors, *first*, a temperature of about 1700 degrees Fahrenheit, during four hours, a load of 150 pounds per square foot being upon them; and *second*, a stream of water forced upon them while still at about the temperature above stated. A column was placed in the chamber roofed by the rock concrete, and it was tested the same way.

The fuel used was seasoned pine wood and the stoking was looked after by a man experienced in a pottery; hence a very even fire was maintained, except at first, on the cinder concrete side, where the blaze began in one corner and spread rather slowly for some time.

The water was supplied from a pump at which 90 pounds pressure was maintained, and was delivered through 200 feet of new cotton hose and a $1\frac{1}{8}$-inch nozzle. Each side was drenched with water while at full temperature, apparently; and the water was thrown as uniformly as possible over the surface to be tested, for the required time. The floors were then flooded on top, and again treated underneath.

Inasmuch as the floors and the column were the only parts submitted for tests, the slight cracking and pitting of the walls and partition need not be detailed.

The column was practically intact, except that a few small pieces of the concrete were washed out where struck by the stream at close range. The metal, however, remained completely covered.

within about 7 inches of the ends on one beam, and about 2 feet from the ends on the other beam. The reinforcing bars were denuded over an area of about 30 square feet near the center; but no cracks developed, and the water poured on top seemed to come down only through the pipe set in for the pyrometer.

On the cinder concrete side, the beams lost only a little of the edges of the covering, not showing the metal at all. There were no cracks on this side either, and the water came down through the pyrometer tube as on the other side. The metal in the slab was bared over an area of about 24 square feet near the center.

During the firing, both chambers were occasionally examined, and no cracking or flaking-off of the concrete could be detected. Hence the water did all the damage that was apparent at the end.

During the test the floors supported the load they were designed to carry; and on the following day the loads were increased to 600 pounds per square foot.

The following is taken from Professor Van Dyck's report:

"The maximum deflection of the stone concrete *before* the application of water, was $2\frac{15}{16}$ inches; *after* application of water, $3\frac{3}{16}$ inches; with normal temperature and original load, $3\frac{1}{16}$ inches; deflection after load of 600 pounds was added, $3\frac{13}{16}$ inches.

"The maximum deflection of the cinder concrete before the application of water, was $6\frac{1}{16}$ inches; after application of water, $6\frac{1}{2}$ inches; with normal temperature and original load, $5\frac{11}{16}$ inches; deflection after a load of 600 pounds was added, 6 inches. These measurements were taken at the center of the roof of each chamber."

Results Shown in Baltimore Fire. Engineers and architects, who made reports on the Baltimore fire of February, 1904, generally state that reinforced concrete construction stood very well much better than terra cotta. Professor Norton, in his report to the Insurance Engineering Experiment Station, says:

"Where concrete floor-arches and concrete-steel construction received the full force of the fire, it appears to have stood well, distinctly better than the terra cotta. The reasons, I believe, are these: The concrete and steel expand at sensibly the same rate, and hence, when heated, do not subject each other to stress; but terra cotta usually expands about twice as fast with increase in temperature as steel, and hence the partitions and floor-arches soon become too large to be contained by the steel members which under ordinary temperature properly enclose them."

METHODS OF MIXING

Two methods are used in mixing concrete—*by hand* and *by machinery*. Good concrete may be made by either method and in either case the concrete should be carefully watched by a good foreman. If a large quantity of concrete is required, it is cheaper to mix it by machinery. On small jobs where the cost of erecting the plant, together with the interest and depreciation, divided by the number of cubic yards to be made, constitute a large item, or if frequent moving is required, it is very often cheaper to mix the concrete by hand. The relative cost of the two methods usually depends upon circumstances, and must be worked out in each individual case.

Mixing by Hand. The placing and handling of materials and arranging the plant are varied by different engineers and contractors. In general the mixing of concrete is a simple operation, but should be carefully watched by an inspector. He should see

(1) That the exact amount of stone and sand are measured out;
(2) That the cement and sand are thoroughly mixed;
(3) That the mass is thoroughly mixed;
(4) That the proper amount of water is used;
(5) That care is taken in dumping the concrete in place;
(6) That it is thoroughly rammed.

Mixing Platform. The mixing platform, which is usually 10 to 20 feet square, is made of 1-inch or 2-inch plank planed on one side and well nailed to stringers, and should be placed as near the work as possible, but so situated that the stone can be dumped on one side of it and the sand on the opposite side. A very convenient way to measure the stone and sand is by the means of bottomless boxes. These boxes are of such a size that they hold the proper proportions of stone or sand to mix a batch of a certain amount. Cement is usually measured by the package, that is, by the barrel or bag, as they contain a definite amount of cement.

Process of Mixing. The method used for mixing the concrete has little effect upon the strength of the concrete, if the mass has been turned a sufficient number of times to thoroughly mix them. One of the following methods is generally used. (Taylor and

(*b*) Cement and sand mixed dry, the stone measured and dumped on top of it, leveled off, and wet, as turned with shovels.

(*c*) Cement and sand mixed into a mortar, the stone placed on top of it, and the mass turned.

(*d*) Cement and sand mixed with water into a mortar which is shoveled on the gravel or stone and the mass turned with shovels.

(*e*) Stone or gravel, sand, and cement spread in successive layers, mixed slightly and shoveled into a mound, water poured into the center, and the mass turned with shovels.

The quantity of water is regulated by the appearance of the concrete. The best method of wetting the concrete is by measuring the water in pails. This insures a more uniform mixture than by spraying the mass with a hose.

Mixing by Machinery. On large contracts the concrete is generally mixed by machinery. The economy is not only in the mixing itself but in the appliances introduced in handling the raw materials and the mixed concrete. If all materials are delivered to the mixer in wheelbarrows, and if the concrete is conveyed away in wheelbarrows, the cost of making concrete is high, even if machine mixers are used. If the materials are fed from bins by gravity into the mixer, and if the concrete is dumped from the mixer into cars and hauled away, the cost of making the concrete should be very low. On small jobs the cost of maintaining and operating the mixer will usually exceed the saving in hand labor and will render the expense with the machine greater than without it.

Machine vs. Hand Mixing. It has already been stated that good concrete may be produced by either machine or hand mixing, if it is thoroughly mixed.

Tests made by the U. S. Government engineers at Duluth, Minn., to determine the relative strength of concrete mixed by hand and mixed by machine (a cube mixer), showed that at 7 days, hand-mixed concrete possessed only 53 per cent of the strength of the machine-mixed concrete; at 28 days, 77 per cent; at 6 months, 84 per cent; and at one year, 88 per cent. Details of these tests are given in Table X.

It should be noted in this connection, that the variations in strength from highest to lowest were greatest in the hand-mixed

Tensile Tests of Concrete*

Age, and Method of Mixing	High	Low	Average
Age 7 Days			
Machine-Mixed Sample	260	243	253
Hand-Mixed Sample	159	113	134
Age 28 Days			
Machine-Mixed Sample	294	249	274
Hand-Mixed Sample	231	197	211
Age 6 Months			
Machine-Mixed Sample	441	345	388
Hand-Mixed Sample	355	298	324
Age One Year			
Machine-Mixed Sample	435	367	391
Hand-Mixed Sample	369	312	343

The mixture tested was composed of 1 part cement and 10.18 parts aggregate.

STEEL FOR REINFORCING CONCRETE

Quality of Reinforcing Steel. Steel for reinforcing concrete is not usually subjected to as severe treatment as ordinary structural steel, as the impact effect is likely to be a little less; but the quality of the steel should be carefully specified. To reduce the cost of reinforced concrete structures, there has been a tendency to use cheap steel. This has resulted in bars being rolled from old railroad rails. These bars are known as rerolled bars and they should always be thoroughly tested before being used. If the bars are rerolled from rails that were made of good material, they should prove to be satisfactory, but if the rails contained poor materials the bars rolled from them will probably be brittle and easily broken by a sudden blow. Many engineers specify that the bars shall be rolled from billets to avoid using any old material.

The grades of steel used in reinforced concrete range from soft to hard, and may be classified under three heads: soft, medium, and hard.

Soft Steel. Soft steel has an estimated strength of 50,000 to 58,000 pounds per square inch. It is seldom used in reinforced concrete.

*(From "Concrete and Reinforced Concrete Construction", by H. A. Reid.)

Medium Steel. Medium steel has an estimated strength of 55,000 to 65,000 pounds per square inch. The elastic limit is from 32,000 to 38,000 pounds per square inch. This grade of steel is extensively used for reinforced concrete work and can be bought in the open market and used with safety.

Hard Steel. Hard steel, better known as *high-carbon steel,* should have an ultimate strength of 85,000 to 100,000 pounds per square inch; and the elastic limit should be from 50,000 to 65,000 pounds per square inch. The hard steel has a greater percentage of carbon than the medium steel, and therefore the yield point is higher. This steel is preferred by some engineers for reinforced concrete work, but it should be thoroughly tested to be sure that it is according to specifications. It is often brittle. This is the grade of steel into which old rails are rolled, but it is also rolled from billets.

Processes of Making Steel. Reinforcing bars are rolled by both the Bessemer and the open-hearth processes. Bars rolled by either process make good reliable steel, but bars rolled by open-hearth process are generally more uniform in quality.

TYPES OF BARS

The steel bars used in reinforcing concrete usually consist of small bars of such shape and size that they may easily be bent and placed in the concrete so as to form a monolithic structure. To distribute the stress in the concrete, and secure the necessary bond between the steel and concrete, the steel required must be supplied in comparatively small sections. All types of the regularly rolled small bars of square, round, and rectangular section, as well as some of the smaller sections of structural steel, such as angles, T-bars, and channels, and also many special rolled bars, have been used for reinforcing concrete. These bars vary in size from $\frac{1}{4}$ inch for light construction, up to $1\frac{1}{2}$ inches for heavy beams, and up to 2 inches for large columns. In Europe plain round bars have been extensively used for many years and the same is true in the United States, but not to the same extent as in Europe; that is,

Square and round bars show about the same adhesive strength, but the adhesive strength of flat bars is far below that of the round and square bars. The round bars are more convenient to handle and easier obtained, and have, therefore, generally been used when plain bars were desirable.

Structural Steel. Small angles, T-bars, and channels have been used to a greater extent in Europe than in this country. They are principally used where riveted skeleton work is prepared for the steel reinforcement; and in this case, usually, it is desirable to have the steel work self-supporting.

Deformed Bars. There are many forms of reinforcing materials on the market, differing from one another in the manner of forming the irregular projections on their surface. The object of all these special forms of bars is to furnish a bond with the concrete, independent of adhesion. This bond formed between the deformed bar and

Fig. 15. Square Twisted Reinforcing Steel Bar
Courtesy of Inland Steel Company

the concrete is usually called a *mechanical bond.* Some of the most common types of bars used are the *square twisted* bar; the *corrugated;* the *Havemeyer;* and the *Kahn.*

Square Twisted Bar. The twisted bar, shown in Fig. 15, was one of the first steel bars shaped to give a mechanical bond with concrete. This type of bar is a commercial square bar twisted while cold. There are two objects in twisting the bar—*first,* to give the metal a mechanical bond with the concrete; *second,* to increase the elastic limit and ultimate strength of the bar. In twisting the bars, usually one complete turn is given the bar in nine or ten diameters of the bar, with the result that the elastic limit of the bar is increased from 40 to 50 per cent, and the ultimate strength is increased from 25 to 35 per cent. These bars can readily be bought already twisted; or, if it is desired, square bars may be bought and twisted on the site of the work.

Corrugated Bar. The "corrugated" bar, which has corrugations as shown in Fig. 16, was invented by Mr. A. L. Johnson,

M. Am. Soc. C. E. These corrugations, or square shoulders, are placed at right angles to the axis of the bar, and their sides

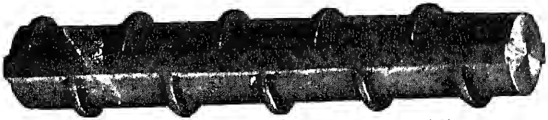


Fig. 16. "Corrugated" Bar for Reinforcement of Concrete
Courtesy of Corrugated Bar Company

make an angle with the perpendicular to the axis of the bars not exceeding the angle of friction between the bar and concrete. These bars are usually rolled from high-carbon steel having an elastic limit of 55,000 to 65,000 pounds per square inch and an ultimate strength cf about 100,000 pounds per square inch. They are also rolled from any desired quality of steel. In size they range from $\frac{1}{4}$ inch to $1\frac{1}{4}$ inches, their sectional area being the same as that of plain bars of the same size. These bars are rolled in both the common types, round and square.

Havemeyer Bar. The Havemeyer bar, Fig. 17, was invented by Mr. J. F. Havemeyer. This has a uniform cross section throughout

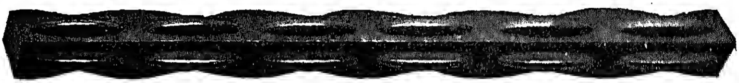


Fig. 17. Havemeyer Bar for Reinforcement of Concrete
Courtesy of Concrete Steel Company

its length. The bonding of the bar to the concrete is uniform at all points, and the entire section is available for tensile strength.

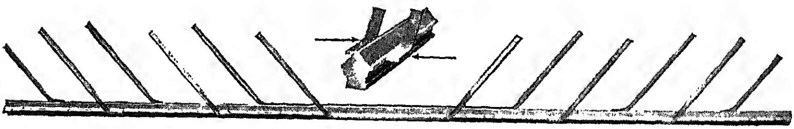


Fig. 18. Kahn Trussed Bar for Reinforcement of Concrete
Courtesy of The Kahn System

Kahn Bar. The Kahn bar, Fig. 18, was invented by Mr. Julius Kahn, Assoc. M. Am. Soc. C. E. This bar is designed with the assumption that the shear members should be rigidly connected to the horizontal members. The bar is rolled with

TABLE XI

Standard Sizes of Expanded Metal

Mesh in Inches	Gage No.	Weight in Lb. per Sq. Ft.	Sectional Area 1 Foot Wide in Sq. In.
3	16	.30	.082
3	10	.625	.177
6	4	.86	.243

as shown in the figure. The thin edges are cut and turned up, and form the shear members. These bars are manufactured in several sizes.

Expanded Metal. Expanded metal, Fig. 19, is made from plain sheets of steel, slit in regular lines and opened into meshes of any desired size or section of strand. It is commercially designated by giving the gage of the steel and the amount of displacement between the junctions of the meshes. The most common manufactured sizes are given in Table XI.

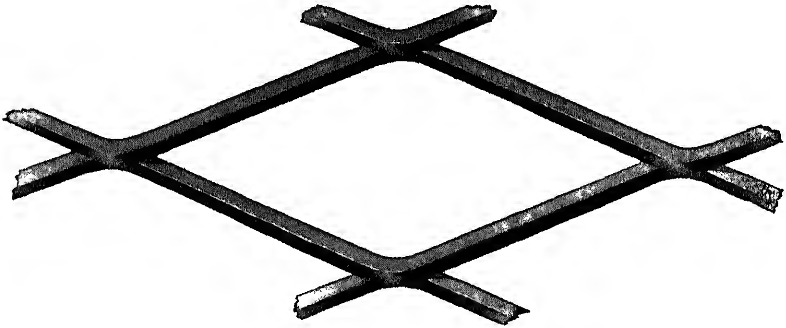


Fig. 19. Example of Expanded Metal Fabric
Courtesy of Northwestern Expanded Metal Company

Steel Wire Fabric. Steel wire fabric reinforcement consists of a netting of heavy and light wires, usually with rectangular meshes. The heavy wires carry the load, and the light ones are used to space the heavier ones. There are many forms of wire fabric on the market.

Table XII is condensed from the handbook of the Cambria Steel Company and gives the standard weights and areas of plain round and square bars as commonly used in reinforced concrete construc-

TABLE XII

Weights and Areas of Square and Round Bars

(One cubic foot of steel weighs 489.6 pounds)

Thickness or Diameter (Inches)	Weight of Square Bar, 1 Foot Long (Pounds)	Weight of Round Bar, 1 Foot Long (Pounds)	Area of Square Bar (Sq. In.)	Area of Round Bar (Sq. In.)	Circum. of Round Bar (Inches)
$\frac{1}{4}$	.213	.167	.0625	.0491	.7854
$\frac{5}{16}$	.332	.261	.0977	.0767	.9817
$\frac{3}{8}$	.478	.376	.1406	.1104	1.1781
$\frac{7}{16}$	.651	.511	.1914	.1503	1.3744
$\frac{1}{2}$	.850	.668	.2500	.1963	1.5708
$\frac{5}{8}$	1.328	1.043	.3906	.3068	1.9635
$\frac{3}{4}$	1.913	1.502	.5625	.4418	2.3562
1	3.400	2.670	1.0000	.7854	3.1416
$1\frac{1}{8}$	4.303	3.379	1.2656	.9940	3.5343
$1\frac{1}{4}$	5.312	4.173	1.5625	1.2272	3.9270
$1\frac{1}{2}$	7.650	6.008	2.2500	1.7671	4.7124
$1\frac{3}{4}$	10.41	8.178	3.0625	2.4053	5.4978
2	13.60	10.68	4.0000	3.1416	6.2832

SPECIFICATIONS FOR REINFORCING BARS.

Process of Manufacture. Steel may be made by either the open-hearth or Bessemer process.

Bars shall be rolled from billets.

Chemical and Physical Properties. The chemical and physical properties of reinforcing bars shall conform to the limits as given in Table XIII.

Chemical Determinations. In order to determine if the material conforms to the chemical limitations prescribed in the above paragraph, analysis shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt or blow of steel, and a certified copy of such analysis shall be furnished to the engineer or his inspector.

Yield Point. For the purpose of these specifications, the yield point shall be determined by careful observation of the drop of the testing machine, or by other equally accurate method.

Form of Specimens. (a) Tensile and bending test specimens of cold-twisted bars shall be cut from the bars after twisting, and shall be tested in full size without further treatment, unless otherwise specified as in (c), in which case the conditions therein stipulated shall govern.

(b) Tensile and bending test specimens may be cut from the

TABLE XIII

Properties Considered	Structural Steel Grade		Hard Grade
	Plain Bars	Deformed Bars	
Phosphorus, maximum Bessemer Open-hearth	 0.10 0.06	 0.10 0.06	 0.10 0.06
Ultimate tensile strength, pounds per square inch.	55,000 to 65,000	55,000 to 65,000	85,000 to 105,000
Yield point, minimum pounds per sq. in.	33,000	33,000	52,000
Elongations, per cent in 8 inches, minimum	1,250,000 tensile str.	1,250,000 tensile str.	1,200,000 tensile str.
Cold bend without fracture: Bars under $\frac{3}{4}$ inch in diameter or thickness	180°, d=1 t.	180°, d=1 t.	180°, d=3 t.
Bars $\frac{3}{4}$ inch in diameter or thickness and over	180°, d=1 t.	180°, d=2 t.	90°, d=3 t.

bars may be planed or turned for a length of at least 9 inches, if deemed necessary by the manufacturer in order to obtain uniform cross section.

(c) If it is desired that the testing and acceptance for cold-twisted bars be made upon the rolled bars before being twisted, the bars shall meet the requirements of the structural steel grade for plain bars given in this specification.

Number of Tests. At least one tensile test and one bending test shall be made from each melt of open-hearth steel rolled, and from each blow or lot of ten tons of Bessemer steel rolled. In case bars differing $\frac{3}{8}$ inch and more in diameter or thickness are rolled from one melt or blow, a test shall be made from the thickest and thinnest material rolled. Should either of these test specimens develop flaws, or should the tensile test specimen break outside of the middle third of its gaged length, it may be discarded and another test specimen substituted therefor. In case a tensile test specimen does not meet the specifications an additional test may be made. The bending test may be made by pressure or by light blows.

Modification in Elongation for Thin and Thick Material. For bars less than $\frac{7}{16}$ inch and more than $\frac{3}{4}$ inch nominal diameter or

thickness, the following modifications shall be made in the requirements for elongation:

(a) For each increase of $\frac{1}{8}$ inch in diameter or thickness above $\frac{3}{4}$ inch, a deduction of 1 shall be made from the specified percentage of elongation.

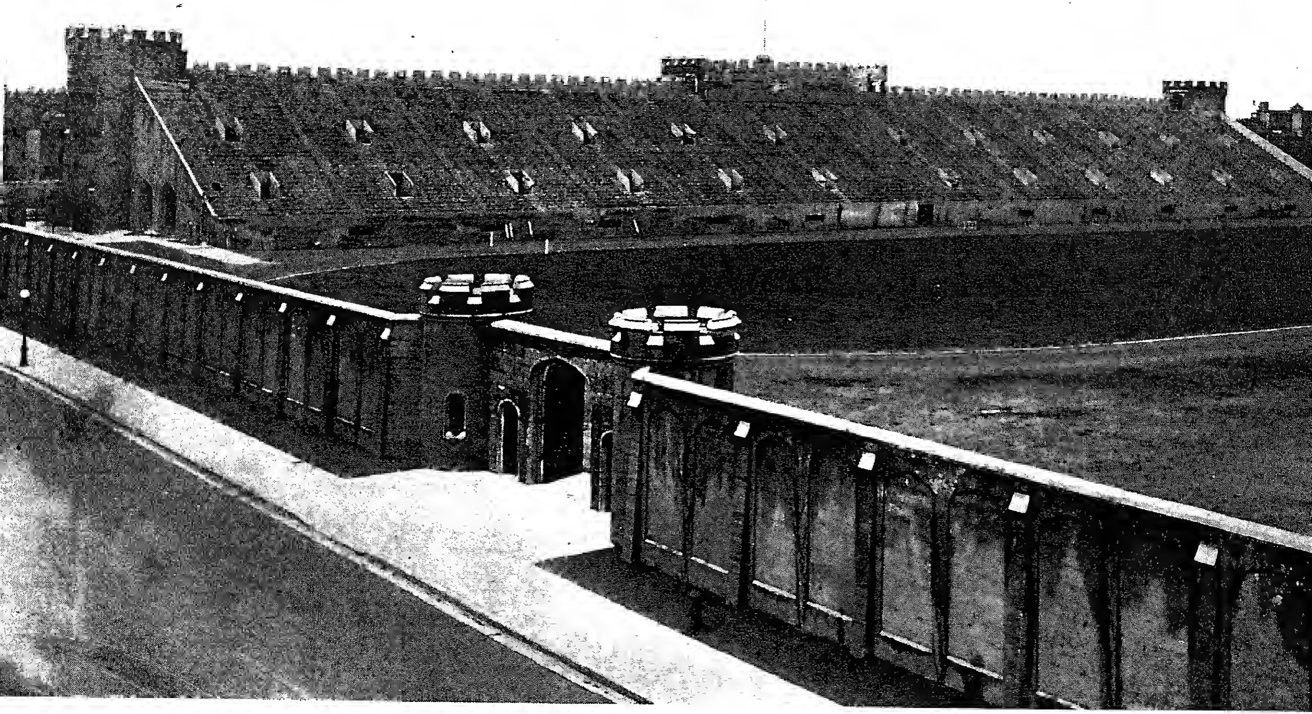
(b) For each decrease of $\frac{1}{16}$ inch in diameter or thickness below $\frac{7}{16}$ inch, a deduction of 1 shall be made from the specified percentage of elongation.

(c) The above modifications in elongation shall not apply to cold-twisted bars.

Number of Twists. Cold-twisted bars shall be twisted cold with one complete twist in a length equal to not more than 10 times the thickness of the bar.

Finish. Material must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

Variation in Weight. Bars for reinforcement are subject to rejection if the actual weight of any lot varies more than 5% over or under the theoretical weight of the lot.



UNIVERSITY OF CHICAGO STADIUM ON STAGG FIELD

This reinforced concrete Grand Stand covers the west side of the field, seats 12,000 people, and the space underneath the seats contains dressing rooms, squash, racquet, and hand-ball courts; and other provisions for athletic games. The reinforced concrete fence in the foreground extends around the entire field.

MASONRY AND REINFORCED CONCRETE

PART II

TYPES OF MASONRY

INTRODUCTION

Definitions. In the following paragraphs, the meanings of various technical terms frequently used in stone masonry are clearly explained:

Arris. Arris is the external edge formed by two surfaces, whether plane or curved, meeting each other.

Ashlar. Ashlar is a style of stone wall built of stones having rectangular faces and with joints dressed so closely that the distance between the general planes of the surfaces of the adjoining stones is one-half inch, or less.

Ax or *Peen Hammer.* A peen hammer is a tool, Fig. 20, which is similar to a double-bladed wood ax. It is used after the stone is rough-pointed, to make drafts along the edges of the stone. For rubble work, and even for squared-stone work, no finer tool need be used.

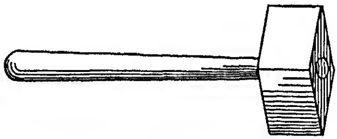


Fig. 20. Ax or Peen Hammer

Backing. Backing is the masonry on the back side of a wall; it is usually of rougher quality than that on the face.

Batter. Batter is the term used to indicate the variation from the perpendicular, of a wall surface. It is usually expressed as the ratio of the horizontal distance to the vertical height. For example, a batter of 1:12 means that the wall has a slope of one inch horizontally to each twelve inches of height.

Bearing Block. The bearing block is a block of stone set in a wall with the special purpose of forming a bearing for a concentrated load, such as the load of a beam.

Bed Joint. A horizontal joint, or one which is nearly perpendicular to the resultant line of pressure, is called a bed joint. (See *Joint.*)

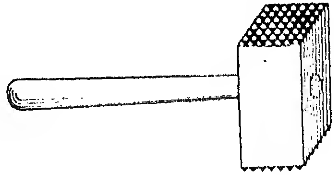


Fig. 21. Bushhammer

Belt Course. A belt course is a horizontal course of stone extending around one or more faces of a building; it is usually composed of larger stones which sometimes project slightly and is, in most instances, employed only for architectural effect.

Bonding. Bonding is the system according to which the stones are arranged so that they are mutually tied together by the overlapping of joints.

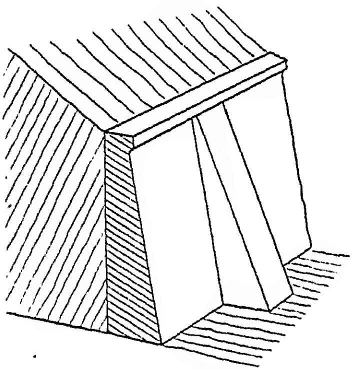
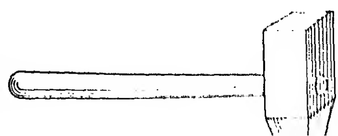


Fig. 22. Buttress

Bushhammering. Bushhammering is a method of finishing stone by which the face of the stone, after being roughly dressed to a surface which is nearly plane, is smoothed still more with a *bushhammer*, Fig. 21. The face of the bushhammer has a large number of small pyramidal points, that, in skillful hands, speedily reduce the surface to a uniformly granular condition.

Buttress. A buttress is a very short projection, Fig. 22, built perpendicular to a main wall which may be subjected to lateral thrust, in order to resist, by compression, the tendency of the wall to tip over. (See *Counterfort.*)

Cavil. A cavil is a tool which has one blunt face, and a pyramidal point at the other end, Fig. 23. It is used for roughly breaking up



forged and ground to a chisel edge, as shown in Fig. 24. It is used for cutting drafts for the edges of stones and is usually driven by a mallet or hammer.

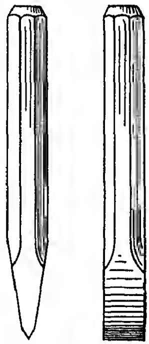


Fig. 24. Chisel

Coping. The coping is the course of stone which caps the top of a wall.

Corbel. A stone projecting from the face of a wall for the purpose of supporting a beam or an arch which extends out from the wall is called a corbel.

Counterfort. A counterfort is a short projection built behind a retaining wall in order to relieve by tension the overturning thrust against the wall. (See *Buttress.*)

Course. A course is a row of stones of equal height laid horizontally along a wall.

Coursed Masonry. Masonry having courses of equal height throughout is termed coursed masonry.

Coursed Rubble. Rubble masonry (see *Rubble*), in which the stones in each course are roughly dressed to nearly a uniform height, is designated as coursed rubble.

Cramp. A cramp is a bar of iron, having the ends bent at right angles, which is inserted in holes and grooves specially cut for it in adjacent stones in order to bind the stones together. When carefully packed with cement mortar, these iron cramps are effectively prevented from rusting.

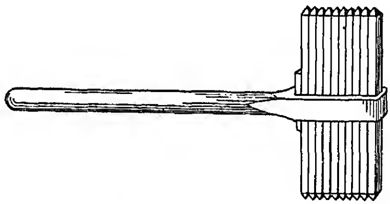


Fig. 25. Crandall

Crandall. A crandall, Fig. 25, is a tool made by fitting a series of steel points into a handle, using a wedge; by means of this device a series of fine picks at the stone are made with each stroke, and the surface is more quickly reduced to a true plane.

Crandalling. Crandalling is the system of dressing stone by which the surface, after having been rough-pointed to a fairly plane surface, is hammered with a crandall, such as is illustrated in Fig. 25.

Dimension Stone. Dimension stone is the cut stone whose precise dimensions in a building are specified in the plans. The term refers to the highest grade of ashlar work.

Dowel. A dowel is a straight bar or pin of iron, copper, or even of stone, which is inserted in two corresponding holes in adjacent stones. The dowels may be vertical across horizontal joints, or horizontal across vertical joints. In the latter case, they are frequently used to tie the stones of a coping or cornice. The extra space between the dowels and the stones should be filled with melted lead, sulphur, or cement grout.

Draft. Draft is the term applied to a line on the surface of a stone which is cut to the breadth of the draft chisel.

Dry-Stone Masonry. Dry-stone masonry is masonry which is put in place without mortar.

Extrados. The extrados is the upper, or outer, surface of an arch, especially the upper curved face of the whole body of voussoirs. (Compare *Intrados.*)

Face. The face is the exposed surface of a wall.

Face Hammer. A face hammer, Fig. 26, is a tool having a hammer face and an ax face. It is used for roughly squaring up stones, either for rubble work or in preparation for finer stone dressing.

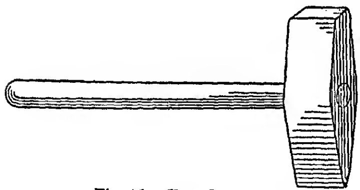


Fig. 26. Face Hammer

Feathers. See *Plugs.*

Footing. The footing is the foundation masonry for a wall or pier, usually composed, in stone masonry, of large stones having a sufficient area so that the pressure upon the subsoil shall not exceed a safe limit, and having sufficient transverse strength to distribute the pressure uniformly over the subsoil.

Grout. Grout is a mixture of cement and sand (usually 1 part cement to 1 or 2 parts sand) made into a very thin mortar so that it will flow freely into interstices left between stones of rough masonry. Grout is used to great advantage in many lines of work.

Header. A header is a stone laid with its greatest dimension perpendicular to the face of a wall. Its purpose is to bond together

Intrados. The intrados is the inner, or under, surface of an arch.

Jamb. The jamb is the vertical surface on either side of an opening left in a wall for a door or window.

Joint. The horizontal and vertical spaces between the stones, which are filled with mortar, are called the joints. When they are horizontal, they are called *bed joints.* Their width or thickness depends on the accuracy with which the stones are dressed. The joint should always have such a width that any irregularity on the surface of a stone shall not penetrate completely through the mortar joint and cause the stones to bear directly on each other, thus producing concentrated pressures and transverse stresses which might rupture the stones. The criterion used by a committee of the American Society of Civil Engineers in classifying different grades of masonry is to make the classification depend on the required thickness of the joint. These thicknesses have been given when defining various grades of stone masonry.

Lintel. The lintel is the stone, iron, wood, or concrete beam covering the opening left in a wall for a door or window.

Natural Bed. The surfaces of a stone parallel to its stratification are called the natural bed.

One-Man Stone. One-man stone is a term used to designate, roughly, the size and weight of stone used in a wall. It represents, approximately, the size of stone which can be readily and continuously handled by one man.

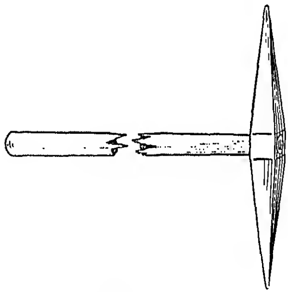


Fig. 27. Pick

Pick. A pick is a tool which roughly resembles an earth pick, but which has two sharp points. It is used like a cavil for roughly breaking up and forming the stones as desired, Fig. 27.

Pitch-Faced Masonry. Pitch-faced masonry, Fig. 28, is masonry in which the edges of the stone are dressed to form a rectangle which lies in a true plane, although the portion of the face between the edges is not plane.

Pitching Chisel. A pitching chisel is a tool which is used with a mallet to prepare pitch-faced masonry. The usual forms are illustrated in Fig. 29.

Plinth. Plinth is another term for *Water Table,* see page 94.

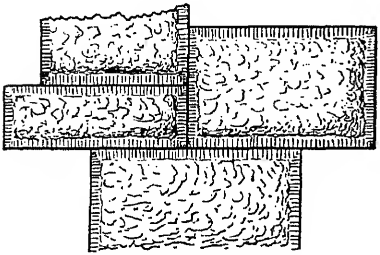


Fig. 28. Pitch-Faced Masonry

Plug. A plug is a truncated wedge, Fig. 30. Corresponding with it are wedge-shaped pieces made of half-round malleable iron. A plug is used in connection with a pair of feathers to split a section of stone uniformly. A row of holes is drilled in a straight line along the surface of the stone, and a plug and pair of feathers are inserted in each hole. The plugs in succession are tapped lightly with a hammer so that the pressure produced by all the plugs is increased as uniformly as possible. When the pressure is uniform, the stone usually splits along the line of the holes without injury to the portion split apart.

Point. A point is a tool made of a bar of steel whose end is ground to a point. It is used in the intermediate stage of dressing an irregular surface which has already been roughly trued up with a face hammer or an ax. For rough masonry, this may be the finishing tool. For higher-grade masonry, such work will be followed by bushhammering, crandalling, etc.

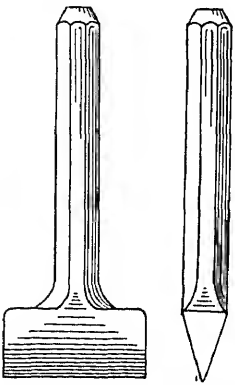


Fig. 29. Pitching Chisel

Pointing. Pointing is the term applied to the process of scraping out the mortar for a depth of an inch or more on the face of a wall after the wall is complete and is supposed to have become compressed to its final form; the joints are then filled with a very rich mortar—say equal parts of cement and sand. Although ordinary brickwork is usually laid by finishing the joints as the work proceeds, it is impossible to prevent some settling of the masonry, which usually squeezes out some of the mortar and

through the cracks into the wall. By scraping out the mortar, which may be done with a hook before it has become thoroughly hard, the joint may be filled with a high grade of mortar which will render it practically impervious to rainwater. The pointing may be done with a mason's trowel, although, for architectural effect, such work is frequently finished off with specially formed tools which will mold the outer face of the mortar into some desired form.

Quarry-Faced Stone. Quarry-faced stone is stone laid in the wall, in the condition in which it comes from the quarry. The term usually applies to stones which have such regular cleavage planes that even the quarry faces are sufficiently regular for use without dressing.

Quoin. A quoin is a stone placed in the corner of a wall so that it forms a header for one face and a stretcher for the other.

Random. Random is the converse of *Coursed Masonry;* masonry which is not laid in courses.

Range. A range is a row or course with the horizontal joints continuous. Range masonry is masonry in which each course has the same thickness throughout, but the different courses vary in thickness.

Riprap. Riprap consists of rough stone, just as it comes from the quarry, which is placed on the surface of an earth embankment.

Rough-Pointing. Rough-pointing is dressing the face of a stone by means of a pick, or perhaps a point, until the surface is approximately plane. This may be the first stage preliminary to finer dressing of the stones.

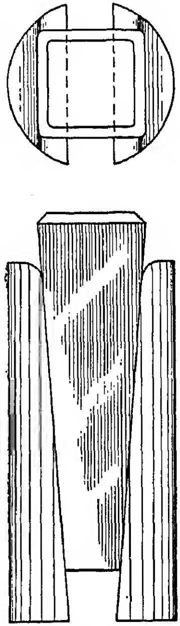


Fig. 30. Plug and Feathers

Rubble. Rubble is the name given masonry composed of rough stones as they come from the quarry, without any dressing other than knocking off any objectionable protruding points. The thickness may be quite variable, and therefore the joints are usually very thick in places.

Slope-Wall Masonry. Slope-wall masonry signifies a type of wall, usually of dry rubble, which is built on a sloping bank of earth

and supported by it, the object of the wall being, chiefly, to protect the embankment against scour.

Spalls. Spalls are small stones and chips, selected according to their approximate fitness, which are placed between the larger, irregular stones in rubble masonry in order to avoid, in places, an excessive thickness of the mortar joint. Specifications sometimes definitely forbid their use.

Squared-Stone Masonry. Squared-stone masonry is masonry in which the stones are roughly dressed so that at the joints the distance between the general planes of the surface of adjoining stones is one-half inch or more.

Stretcher. A stretcher is a stone which is placed in the wall so that its greatest dimension is parallel with the wall.

Stringcourse. A stringcourse is a course of stone or brick, running horizontally around a building, whose sole purpose is architectural effect. (See *Belt Course.*)

Template. A template is a wooden form used as a guide in dressing stones to some definite shape, as illustrated in Figs. 33 and 34.

Two-Man Stone. Two-man stone is a rather indefinite term applied to a size and weight of stone which cannot be readily handled except by two men. The term has a significance in planning the masonry work.

Voussoir. A voussoir is one of the tapering or wedge-shaped pieces of which an arch or vault is composed. The middle one is usually called the keystone.

Water Table. The water table is a course of stone which projects slightly from the face of the wall, and is usually laid at the top of the foundation wall. Its function is chiefly architectural, although, as its name implies, it is supposed to divert the water which might drain down the wall of a building, and to prevent it from following the face of the foundation wall.

Wood Brick. Wood brick is the name for a block of wood placed in a wall in a situation where it will later be convenient to drive nails or screws. Such a block is considered preferable to the plan of subsequently drilling a hole and inserting a plug of wood into which the screws or nails may be driven, since such a plug may act as a wedge and crack the masonry.

Classification of Dressed Stones. Stone masonry is classified according to the shape of the stones, and also according to the quality and accuracy of the dressing of the joints so that the joints may be close. The definitions of these various kinds of stonework have already been given in the previous pages, and therefore will not be repeated here; but the classification will be repeated in the order of the quality and usual relative cost of the work.

The term *rubble* is usually applied to stone masonry on which but little work has been done in dressing the stones, although the cleavage planes may be such that very regular stones may be produced with very little work. Rubble masonry usually has joints which are very irregular in thickness. In order to reduce the amount of clear mortar which otherwise might be necessary in places between the stones, small pieces of stone called *spalls* are placed between the larger stones. Such masonry is evidently largely dependent upon the shearing and tensile strength of the mortar and is therefore comparatively weak. *Random rubble,* Fig. 31, which has joints that are not in general horizontal or vertical, or even approximately so, must be considered as a weak type of masonry. In fact, the real strength of such walls, which are frequently built for architectural effect, depends on the backing, to which the facing stones are sometimes secured by cramps.

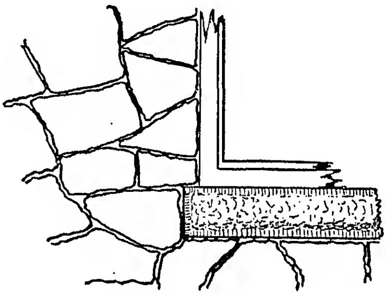


Fig. 31. Random Rubble

The next grade in quality is *squared-stone masonry,* which refers only to the accuracy in dressing the joints and may be applied to *coursed, range,* and *random* work. The term *ashlar* refers both to the rectangular shape of the stone and the accuracy of dressing the joints; it may be applied to *coursed, range,* and *random* work.

Cutting and Dressing Stone. Many of the requirements and methods of stone dressing have already been stated in the definitions given above. Frequently a rock is so stratified that it can be split up into blocks whose faces are so nearly parallel and perpendicular

that in building a substantial wall with comparatively close joints the stones may be used with little or no dressing. On the other hand, an igneous rock such as granite must be dressed to a regular form.

Rectangular Blocks. The first step in making rectangular blocks from any stone is to decide from its stratification, if any, or its cleavage planes, how the stone may be dressed with the least labor in cutting. The stone is then marked in straight lines with some form of marking chalk, and drafts are cut with a drafting chisel so as to give a rectangle whose four lines lie all in one plane. The other faces are then dressed off with as great accuracy as is desired, so that they are perpendicular, or parallel, to this plane. For squared-stone masonry, and especially for ashlar masonry, the drafts should be cut for the bed joints, and the surface between the drafts on any face should be worked down to a true plane, or nearly so. The bed joints should be made slightly concave rather than convex, but the concavity should be very slight. If the surface is very convex, there is danger that the stones will come in contact with each other and produce a concentration of pressure, unless the joints are made undesirably thick. If they are very concave, there is a danger of developing transverse stresses in the stones, which might cause a rupture. The engineer or contractor must be careful to see that the bed joints are made truly perpendicular to the face. Careless masons will sometimes use the stones in the form of truncated wedges, as illustrated in Fig. 32. Such masonry, when finished, may look almost like ashlar; but such a wall is evidently very weak, even dangerously so.

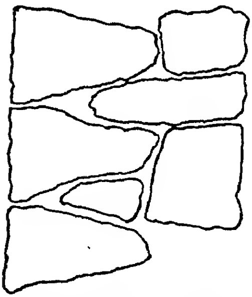


Fig. 32. Defective Work

Cylindrical Surface. To produce a cylindrical surface on a stone, a draft must be cut along the stone, which shall be parallel with the axis of the cylinder, Fig. 33. A template made with a curve of the desired radius, and with a guide which runs along the draft, may be used in cutting down the stone to the required cylindrical form. A circular template, swung around a point which may

although such work is now usually done in a lathe instead of by hand.

Warped Surface. To make a warped surface or helicoidal surface, a template must be made, as in Fig. 34, by first cutting two drafts which shall fit a template made as shown in the figure. After these two drafts are cut, the surface between them is dressed down to fit a straightedge, which is moved along the two drafts and perpendicular to them. Such stonework is very unusual, and almost its only application is in the making of oblique or helicoidal arches.

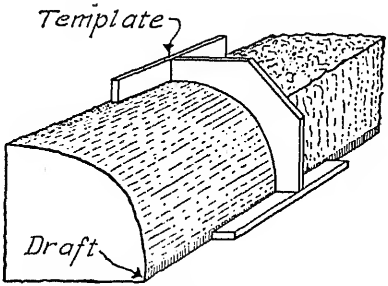


Fig. 33. Template for Cutting Cylindrical Surface

Economical Size of Blocks. The size of the blocks has a very great influence on the cost of dressing the stones per cubic yard of masonry. For example, to quote a very simple case, a stone 3 feet long, 2 feet wide, and 18 inches high has 12 square feet of bed joints, 6 square feet of end joints, and 4.5 square feet of facing, and contains 9 cubic feet of masonry. If the stones are 18 inches long, 1 foot wide, and 9 inches high—just one-half of each dimension—the area of each kind of dressed joint is one-fourth that in the case of the larger stones, but the volume of the masonry is only one-eighth. In other words, for stones of similar shape, increasing the size increases the area of dressing in proportion to the square of the dimensions, but it also increases the volume in proportion to the cube of the dimensions. Therefore large stones are far more economical than small stones, so far as the cost of dressing is a factor.

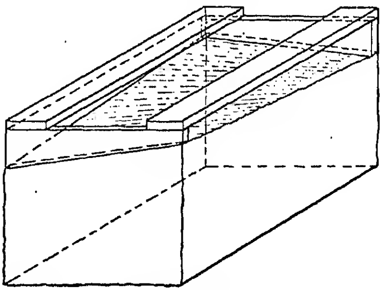


Fig. 34. Template for Warped-Surface Cutting

The size of stones, the thickness of courses, and the type of masonry should depend largely on the product of the quarry to be

less will be the area to be dressed per cubic foot or yard of masonry. On the other hand, the size of blocks which can be broken out from a quarry of stratified rock, such as sandstone or limestone, is usually fixed somewhat definitely by the character of the quarry itself. The stratification reduces very greatly the work required, especially on the bed joints. But since the stratification varies, even in any one quarry, it is generally most economical to use a stratified stone for random masonry, while granite can be cut for coursed masonry at practically the same expense as for stones of variable thickness.

Cost of Dressing Stone. Although, as explained above, the cost of dressing stone should properly be estimated by the square foot of surface dressed, most figures which are obtainable give the cost per cubic yard of masonry, which practically means that the figures are applicable only to stones of the average size used in that work. A few figures are here quoted from Gillette's "Handbook of Cost Data":

(*a*) *Hand Dressing*—Wages, 75 cents per hour. Soft, 38 to 45 cents; medium, 60 to 78 cents; hard, \$1.12 to \$1.20 per square foot of surface dressed.
(*b*) *Hand Dressing*—Wages, \$6.00 per day. Limestone, bushhammered, 50 cents per square foot.
(*c*) *Hand Dressing Limestone*—32 square feet of beds and joints per 8-hour day (or 4 square feet per hour); wages, 75 cents per hour, or 19 cents per square foot.
(*d*) *Hand Dressing Granite*—For $\frac{1}{2}$-inch joints, 50 cents per square foot.
(*e*) *Sawing Slabs by Machinery*—Costs approximately 30 cents per square foot.

Constructive Features. *Bonding.* It is a fundamental principle of masonry construction that vertical joints, either longitudinal or lateral, should not be continuous for any great distance. Masonry walls—except those of concrete blocks—are seldom or never constructed entirely of single blocks which extend clear through the wall. The wall is essentially a double wall which is frequently connected by headers. These break up the continuity of the longitudinal vertical joints. The continuity of the lateral vertical joints is broken up by placing the stones of an upper course over the joints in the course below. Since the headers are made of the same quality of stone (or brick) as the face masonry, while the backing is of comparatively inferior quality, it costs more to put in numerous headers, although strength is sacrificed by neglect to do so. For the best

TABLE XIV

Mortar per Cubic Yard of Masonry

Grade of Masonry	Volume of Mortar per Cubic Yard of Masonry
Ashlar	1 to 2 cubic feet
Squared-Stone	4.5 to 7 cubic feet
Rubble	5.5 to 9 cubic feet

One-fourth or one-fifth is a more usual ratio. Cramps and dowels are merely devices to obtain a more efficient bonding. An inspector must guard against the use of blind headers, which are short blocks of stone (or brick), which have the same external appearance on the finished wall, but which furnish no bond. After an upper course has been laid, it is almost impossible to detect them.

Amount of Mortar. For the same reasons given when discussing the relation of size of stones to amount of dressing required, more mortar per cubic yard of masonry is needed for small stones than for large. The larger and rougher joints, of course, require more mortar per cubic yard of masonry. In Table XIV are given figures which, for the above reasons, are necessarily approximate; the larger amounts of mortar represent the requirements for the smaller sizes of stone, and *vice versa.*

The stones should be thoroughly wetted before laying them in the wall, so that they will not absorb the water in the mortar and ruin it before it can set. It is very important that the bed joints be thoroughly flushed with mortar. All vertical joints should likewise be tightly filled with mortar.

Allowable Unit Pressures. In estimating such quantities, the following considerations must be kept in mind:

(1) The accuracy of the dressing of the stone, particularly the bed joints, has a very great influence.

(2) The strength is largely dependent on that of the mortar.

(3) The strength is so little dependent on that of the stone itself that the strength of the stone cannot be considered a guide to the strength of the masonry. For example, masonry has been known to fail under a load not more than five per cent of the ultimate crushing strength of the stone itself.

(4) The strength of a miniature or small-scale prism of masonry is only a guide to the strength of large prisms. The ultimate strength of these is beyond the capacity of testing machines.

(5) So much depends on the workmanship, that in any structure where

Judging from the computed pressures now carried by noted structures, and also from the pressures sustained by piers, etc., which have shown distress and have been removed, it is evident that, assuming good workmanship, the allowable pressure on masonry is as follows:

Granite Ashlar	up to 400 pounds per sq. inch
Limestone or Sandstone Ashlar	up to 300 pounds per sq. inch
Squared-Stone	up to 250 pounds per sq. inch
Rubble	up to 140 pounds per sq. inch

Somewhat larger pressures may be allowed on the different grades of stone masonry when Portland cement is used in the mortar instead of common lime.

Cost of Stone Masonry. The total cost is a combination of several very variable items as follows:

(1) Value of quarry privilege
(2) Cost of stripping superincumbent earth or disintegrated rock
(3) Cost of quarrying
(4) Cost of dressing
(5) Cost of transportation (teaming, railroad, etc.), from quarry to site of works
(6) Cost of mortar
(7) Cost of centering, scaffolding, derricks, etc.
(8) Cost of laying
(9) Interest and depreciation on plant
(10) Superintendence

Some of the above items may be practically nothing, in cases. The cost of some of the items has already been discussed. The cost of many items is so dependent on local conditions and prices that the quotation of the cost of definite jobs would have but little value and might even be deceptive. The following very general values may be useful to give a broad idea of the cost of stone masonry:

Rubble and Masonry in Mortar	$3.00 to $ 5.00 per cubic yard
Squared-Stone Masonry	6.00 to 10.00 per cubic yard
Dimension Stone, Granite Ashlar	up to 60.00 per cubic yard

BRICK MASONRY

Many of the terms employed in stone masonry as well as the directions for properly doing the work are equally applicable to brick masonry and, therefore, will not be here repeated. The follow-

Bonding Used in Brick Masonry. Some of the principles involved in the effect of bonding on the strength of a wall have

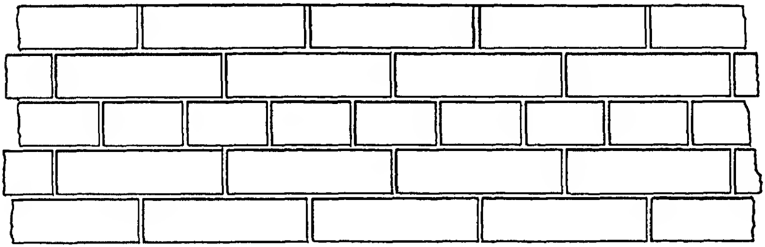


Fig. 35. Common Bond

already been discussed. The other consideration is that of architectural appearance. The common method of bonding, Fig. 35, is to lay five or six courses of brick entirely as stretchers, then a course

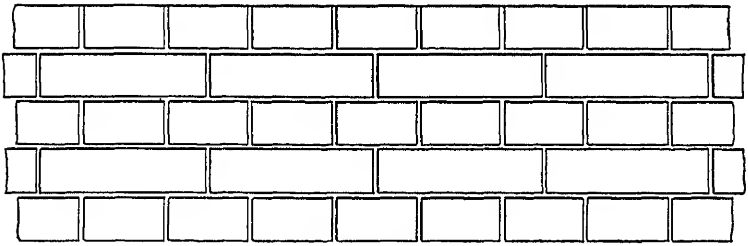


Fig. 36. English Bond

of brick will be laid entirely as headers. There is probably some economy in the work required of a bricklayer in following this policy. The so-called *English bond,* Fig. 36, consists of alternate courses of

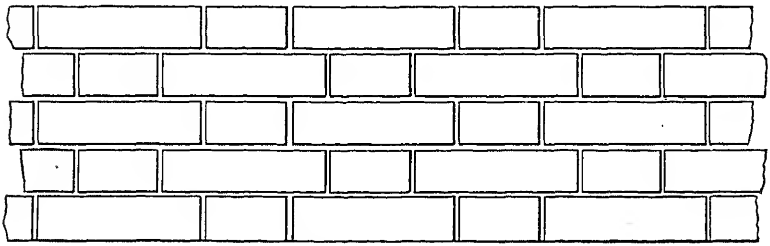


Fig. 37. Flemish Bond

headers and stretchers. If the face bricks are of better quality than those used in the backing of the wall, this system means that

TABLE XV

Quantities of Brick and Mortar

Kind of Brick	Size (Inches)	Thickness of Joints (Inches)	No. of Brick per Cubic Yard	Mortar (Cubic Yard)	
				Per Cubic Yard of Masonry	Per 1,000 Brick
Common brick	$8\frac{1}{4} \times 4 \times 2\frac{1}{4}$	$\frac{1}{2}$	430	.34	.80
Common brick	$8\frac{1}{4} \times 4 \times 2\frac{1}{4}$	$\frac{1}{4}$	516	.21	.40
Pressed brick	$8\frac{3}{8} \times 4\frac{1}{8} \times 2\frac{1}{4}$	$\frac{1}{8}$	544	.11	.21

tainly not an economical way of using the facing brick. The *Flemish bond*, Fig. 37, employs alternate headers and stretchers in each course, and also disposes of the vertical joints so that there is a definite pattern in the joints, which has a pleasing architectural effect.

Constructive Features. On account of the comparatively high absorptive power of brick, it is especially necessary that they shall be thoroughly soaked with water before being laid in the wall. An excess of water can do no harm, and will further insure the bricks being clean from dust, which would affect the adhesion of the mortar. It is also important that the brick shall be laid with what is called a shove joint. This term is even put in specifications, and has a definite meaning to masons. It means that, after laying the mortar for the bed joints, a brick is placed with its edge projecting somewhat over that of the lower brick and is then pressed down into the mortar and, while still being pressed down, is *shoved* into its proper position. In this way is obtained a proper adhesion between the mortar and the brick.

The thickness of the mortar joint should not be over one-half inch; one-fourth inch, or even less, is far better, since it gives stronger masonry. It requires more care to make thin joints than thick joints and, therefore, it is very difficult to obtain thin joints when masons are paid by piecework. Pressed brick fronts are laid with joints of one-eighth inch or even less, but this is considered high-grade work and is paid for accordingly.

Strength of Brickwork. As previously stated with respect to stone masonry, the strength of brick masonry is largely dependent upon the strength of the mortar; but, unlike stone masonry, the strength of brick masonry is, in a much larger proportion, dependent

brick masonry has been determined by a series of tests, to vary from 1,000 to 2,000 pounds per square inch, using lime mortar; and from 1,500 to 3,000 pounds per square inch, using cement mortar—the variation in each group (for the same kind of mortar) depending on the quality of the brick. A large factor of safety, perhaps 10, should be used with such figures.

Methods of Measuring Brickwork. There has been a considerable variation in the methods of measuring brickwork, due to local trade customs, but the general practice now is to measure brickwork by the 1,000 bricks actually laid in the wall. Owing to the variations in size of bricks, no rule for volume of laid brick can be exact. For bricks that measure $8\frac{3}{4}$ inches by $4\frac{1}{4}$ inches by $2\frac{3}{4}$ inches the following scale is a fair average:

7 bricks to a superficial foot for 4-in. wall = 40 lb.
14 bricks to a superficial foot for 9-in. wall = 94 lb.
21 bricks to a superficial foot for 13-in. wall = 121 lb.
28 bricks to a superficial foot for 18-in. wall = 168 lb.
35 bricks to a superficial foot for 22-in. wall = 210 lb.

Common hand-burned bricks weigh from 5 to 6 pounds each. One thousand bricks, closely stacked, occupy about 56 cubic feet. Table XV shows the quantities of brick and mortar for both common and pressed brick.

Cost of Brickwork. A laborer should handle 2,000 brick per hour in loading them from a car to a wagon. If they are not unloaded by dumping, it will require as much time again to unload them. A mason should lay an average of 1,200 brick per 8-hour day on ordinary wall work. For large, massive foundation work with thick walls, the number should rise to 3,000 per day. On the other hand, the number may drop to 200 or 300 on the best grade of pressed-brick work. About one helper is required for each mason. Masons' wages vary from 50 to 75 cents per hour; helpers' wages are about one-half as much.

Impermeability. As previously stated, brick is very porous; ordinary cement mortar is not water-tight; and, therefore, when it is desirable to make brick masonry impervious to water, some special method must be adopted, as described in Part I, under the

page 68, Part I). The Sylvester wash has frequently been used as a preventive, and with fairly good results. Diluted acid has been used successfully to remove the efflorescence. These methods have been described in Part I.

Brick Piers. A brick pier, as a general rule, is the only form of brickwork that is subjected to its full resistance. Sections of walls under bearing plates, also, receive a comparatively large load; but only a few courses receive the full load and, therefore, a greater unit stress may be allowed than for piers.

Kidder gives the following formulas for the safe strength of brick piers exceeding 6 diameters in height:

$$\left.\begin{array}{ll}\text{Piers laid with rich lime mortar} & \\ (a)\qquad \text{Safe load, lb. per sq. in.}=110-5\,\dfrac{H}{D} & \\ \text{Piers laid with 1:2 natural cement mortar} & \\ (b)\qquad \text{Safe load, lb. per sq. in.}=140-5\tfrac{1}{2}\,\dfrac{H}{D} & \\ \text{Piers laid with 1:3 Portland cement mortar} & \\ (c)\qquad \text{Safe load, lb. per sq. in.}=200-6\,\dfrac{H}{D} & \end{array}\right\} \quad (1)$$

In the above formulas, H is the height of the column in feet and D is the diameter of the column in feet.

For example, a column 16 feet in height and $1\frac{3}{4}$ feet square, laid with rich lime mortar, may be subjected to a load of 65 pounds per square inch, or 9,360 pounds per square foot; for a 1:2 natural cement mortar, 90 pounds per square inch, or 12,960 pounds per square foot; and for a 1:3 Portland cement mortar, 146 pounds per square inch, or 20,914 pounds per square foot.

The building laws of some cities require a bonding stone spaced every 3 to 4 feet, when brick piers are used. This stone is 5 to 8 inches thick, and is the full size of the pier. Engineers and architects are divided in their opinion as to the results obtained by using the bonding stone.

CONCRETE MASONRY

Concrete is extensively used for constructing the many different types of foundations, retaining walls, dams, culverts, etc. The ingredients of which concrete is made, the proportioning and the methods of mixing these materials, etc., have been discussed in

Part I. Methods of mixing and handling concrete by machinery will be discussed in Part IV. Various details of the use of concrete in the construction of foundations, etc., will be discussed during the treatment of the several kinds of work.

RUBBLE CONCRETE

Advantages over Ordinary Concrete. Rubble concrete includes any class of concrete in which large stones are placed. The chief use of this concrete is in constructing dams, lock walls, breakwaters, retaining walls, and bridge piers.

The cost of rubble concrete in large masses should be less than that of ordinary concrete, as the expense of crushing the stone used as rubble is saved, and each large stone replaces a portion of cement and aggregate; therefore, this portion of cement is saved, as well as the labor of mixing it. The weight of a cubic foot of stone is greater than that of an equal amount of ordinary concrete, because of the pores in the concrete; the rubble concrete is therefore heavier, which increases its value for certain classes of work. In comparing rubble concrete with rubble masonry, the former is usually found cheaper because it requires very little skilled labor. For walls 3 or 3½ feet thick, the rubble masonry will usually be cheaper, owing to the saving in forms.

Proportion and Size of Stone. Usually the proportion of rubble stone is expressed in percentage of the finished work. This percentage varies from 20 to 65 per cent. The percentage depends largely on the size of the stone used, as there must be nearly as much space left between small stones as between large ones. The percentage therefore increases with the size of the stones. When "one-man" or "two-man" rubble stone is used, about 20 per cent to 25 per cent of the finished work is composed of these stones. When the stones are large enough to be handled with a derrick, the proportion is increased to about 33 per cent; and to 55 per cent, or even 65 per cent, when the rubble stones average from 1 to 2¼ cubic yards each.

The distance between the stones may vary from 3 inches to 15 or 18 inches. With a very wet mixture of concrete, which is generally used, the stones can be placed much closer than if a dry mixture is used. With the latter mixture, the space must be sufficient to allow the concrete to be thoroughly rammed into all of the

crevices. Specifications often state that no rubble stone shall be placed nearer the surface of the concrete than 6 to 12 inches.

Rubble Masonry Faces. The faces of dams are very often built of rubble, ashlar, or cut stone, and the filling between the faces made of rubble concrete. For this style of construction, no forms are required. For rubble concrete, when the faces are not constructed of stone, wooden forms are constructed as for ordinary concrete.

Comparison of Quantities of Materials. The mixture of concrete used for this class of work is often 1 part Portland cement, 3 parts sand, and 6 parts stone. The quantities of materials required for one yard of concrete, according to Table VIII, Part I, are 1.05 bbls. cement, 0.44 cu. yd. sand, and 0.88 cu. yd. stone. If rubble concrete is used, and if the rubble stone laid averages 0.40 cubic yard for each yard of concrete, then 40 per cent of the cubic contents is rubble, and each of the other materials may be reduced 40 per cent. Reducing these quantities gives $1.05 \times 0.60 = 0.63$ bbl. of cement; $0.44 \times 0.60 = 0.26$ cu. yd. sand; and $0.88 \times 0.60 = 0.53$ cu. yd. of stone, per cubic yard of rubble concrete.

The construction of a dam on the Quinebaug River is a good example of rubble concrete. The height of the dam varies from 30 to 45 feet above bed rock. The materials composing the concrete consist of bank sand and gravel excavated from the bars in the bed of the river. The rock and boulders were taken from the site of the dam, and were of varying sizes. Stones containing 2 to $2\frac{1}{2}$ cubic yards were used in the bottom of the dam, but in the upper part of the dam smaller stones were used. The total amount of concrete used in the dam was about 12,000 cubic yards. There was $1\frac{1}{2}$ cubic yards of concrete for each barrel of cement used. The concrete was mixed wet, and the large stones were so placed that no voids or hollows would exist in the finished work.

DEPOSITING CONCRETE UNDER WATER

Methods. In depositing concrete under water, some means must be taken to prevent the separation of the materials while passing through the water. The three principal methods are as follows:

Buckets. For depositing concrete by the first method, special buckets are made with a closed top and hinged bottom. Concrete deposited under water must be disturbed as little as possible and, in tipping a bucket, the material is apt to be disturbed. Several different types of buckets with hinged bottoms have been devised to open automatically when the place for depositing the concrete has been reached. In one type, the latches which fasten the trap-doors are released by the slackening of the rope when the bucket reaches the bottom, and the doors are open as soon as the bucket begins to ascend. In another type, in which the handle extends down the sides of the bucket to the bottom, the doors are opened by the handles sliding down when the bucket reaches the bottom. The doors are hinged to the sides of the bucket and, when opened, permit the concrete to be deposited in one mass. In depositing concrete by this means, it is found rather difficult to place the layers uniformly and to prevent the formation of mounds.

Bags. This method of depositing concrete under water is by means of open-woven bags or paper bags, two-thirds to three-quarters filled. The bags are sunk in the water and placed in courses—if possible, header and stretcher system—arranging each course as laid. The bagging is close enough to keep the cement from washing out and, at the same time, open enough to allow the whole to unite into a compact mass. The fact that the bags are crushed into irregular shapes which fit into each other tends to lock them together in a way that makes even an imperfect joint very effective. When the concrete is deposited in paper bags, the water quickly soaks the paper; but the paper retains its strength long enough for the concrete to be deposited properly.

Tubes. The third method of depositing concrete under water is by means of long tubes, 4 to 14 inches in diameter. The tubes extend from the surface of the water to the place where the concrete is to be deposited. If the tube is small, 4 to 6 inches in diameter, a cap is placed over the bottom, the tube filled with concrete and lowered to the bottom. The cap is then withdrawn, and as fast as the concrete drops out of the bottom, more is put in at the top of the tube, and there is thus a continuous stream of concrete deposited.

When a large tube is used to deposit concrete in this manner, it will be too heavy to handle conveniently if filled before being

lowered. The foot of the tube is lowered to the bottom, and the water rises into the chute to the same level as that outside; and into this water the concrete must be dumped until the water is wholly replaced or absorbed by the concrete. This has a tendency to separate the cement from the sand and gravel, and will take a yard or more of concrete to displace the water in the chute. There is a danger that this amount of badly washed concrete will be deposited whenever it is necessary to charge the chute. This danger occurs not only when the charge is accidentally lost, but whenever the work is begun in the morning, or at any other time. Whenever the work is stopped, the charge must be allowed to run out, or it would set in the tube. The tubes are usually charged by means of wheelbarrows, and a continuous flow of concrete must be maintained. When the chute has been filled, it is raised slowly from the bottom, allowing a part of the concrete to run out in a conical heap at the foot.

This method has also been used for grouting stone. In this case, a 2-inch pipe, perforated at the bottom, is used. The grout, on account of its great specific gravity, is sufficient to replace the water in the interstices between the stones, and firmly cement them into a mass of concrete. A mixture of one part cement and one part sand is the leanest mixture that can be used for this purpose, as there is a great tendency for the cement and sand to separate.

CLAY PUDDLE

Clay puddle consists of clay and sand made into a plastic mass with water. It is used principally to fill cofferdams, and for making embankments and reservoirs water-tight.

Quality of Clay. Opaque clays with a dull, earthy fracture, of an argillaceous nature, which are greasy to the touch, and which readily form a plastic paste when mixed with water, are the best clays for making puddle. Large stones should be removed from the clay, and it should also be free from vegetable matter. Sufficient sand and water should be added to make a homogeneous mass. If too much sand is used, the puddle will be permeable; and if too little is used, the puddle will crack by shrinkage in drying. It is very important that clay for making puddle should show great cohesive power

A simple test to find the cohesive property can easily be made. A small quantity of the clay is mixed with water and made into a roll about 1 inch in diameter and 8 to 10 inches long; and if, on being suspended by one end while wet, it does not break, the cohesive strength is ample. The test to find its water-retaining properties is made by mixing up 1 or 2 cubic yards of the clay with water, making it into a homogeneous plastic mass. A round hole is made in the top of the mass, large enough to hold 4 or 5 gallons of water. The hole is filled with water, and the top covered and left 24 hours; when the cover is removed, the properties of the clay will be indicated by the presence or absence of water.

Puddling. The clay should be spread in layers about 3 inches thick and well chopped with spades, aided by the addition of sufficient water to reduce it to a pasty condition. Water should be given a chance to pass through freely as the clay is being mixed. The different layers, as they are mixed, should be bonded together by the spade passing through the upper layer into the under layer. The test for thorough puddling is that the spade will pass through the layer with ease, which it will not do if there are any hard lumps.

When a large amount of puddle is required, harrows are sometimes used instead of spades. Each layer of clay is thoroughly harrowed, aided by being sprinkled freely with water, and is then rolled with a grooved roller to compact it.

Puddle, when finished, should not be exposed to the drying action of the air, but covered with dry clay or sand.

FOUNDATIONS

PRELIMINARY WORK

Importance of Foundations. It would be impossible to overemphasize the importance of foundations, because the very fact that the foundations are underground and out of sight detracts from the consideration that many will give to the subject. It is probably true that a yielding of the subsoil is responsible for a very large proportion of the structural failures which have occurred. It is also true that many failures of masonry, especially those of arches, are considered as failures of the superstructure, because of breaks occurring in the masonry of the superstructure, which have really

been due, however, to a settlement of the foundations, resulting in unexpected stresses in the superstructure. It is likewise true that the design of foundations is one which calls for the exercise of experience and broad judgment, to be able to interpret correctly such indications as are obtainable as to the real character of the subsoil and its probable resistance to concentrated pressure.

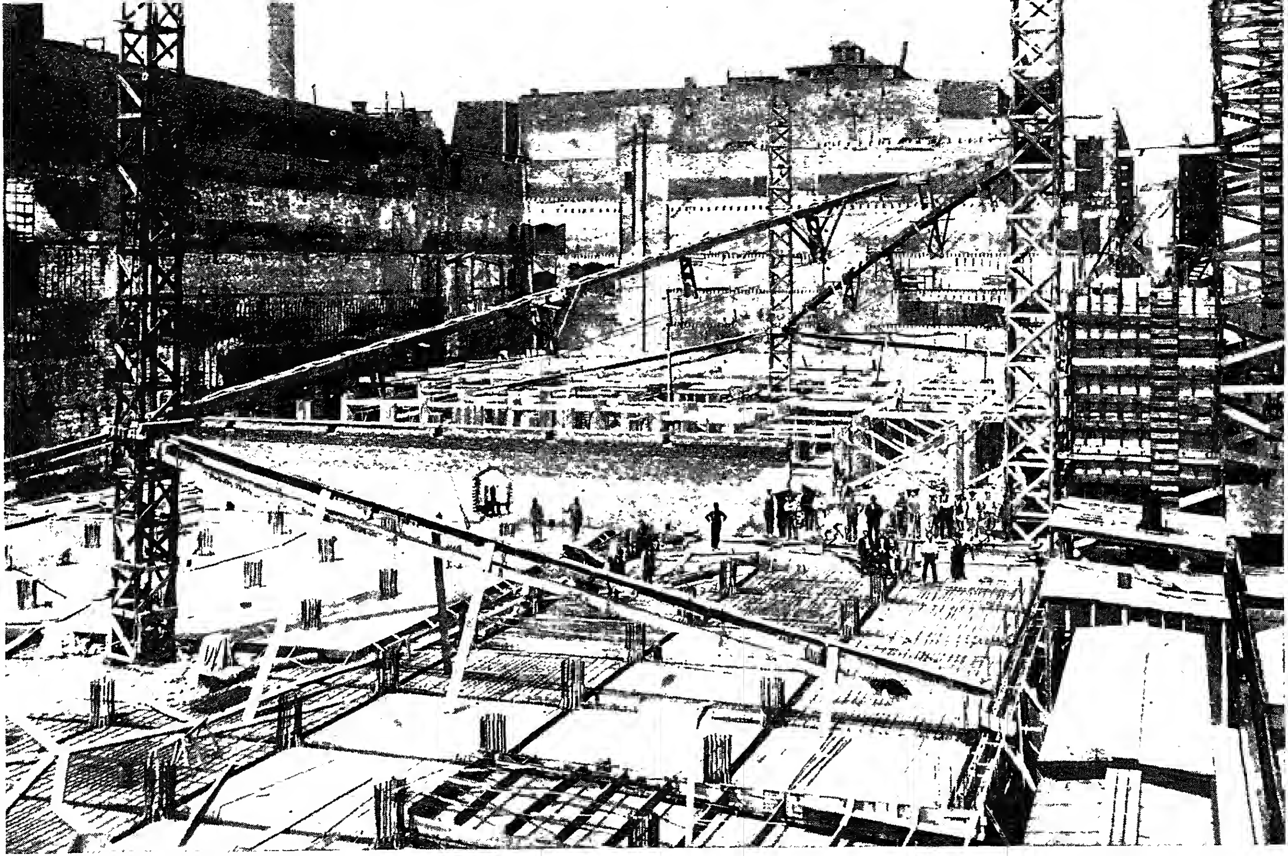
CHARACTER OF SOIL

Classification of Subsoils. The character of soil on which it may be desired to place a structure varies all the way from the most solid rock to that of semi-fluid soils whose density is but little greater than that of water. The gradation between these extremes is so uniform that it is practically impossible to draw a definite line between any two grades. It is convenient, however, to group subsoils into three classes, the classification being based on the method used in making the foundation. These three classes of subsoils are: firm; compressible; and semi-fluid.

Firm Subsoils. These comprise all soils which are so firm, at least at some reasonably convenient depth, that no treatment of the subsoil, or any other special method, needs to be adopted to obtain a sufficiently firm foundation. This, of course, practically means that the soil is so firm that it can safely withstand the desired unit pressure. It also means that a soil which might be classed as firm soil for a light building should be classed as compressible soil for a much heavier building. It frequently happens that the top layers must be removed from rock because the surface rock has become disintegrated by exposure to the atmosphere. Nothing further needs to be done to a subsoil of this kind.

Compressible Subsoils. These include soils which might be considered as firm soils for light buildings, such as dwelling houses, but which could not withstand the concentrated pressure that would be produced, for example, by the piers or abutments of a bridge. Such soils may be made sufficiently firm by methods described later.

Semi-Fluid Subsoils. These are soils such as are frequently found on the banks or in the beds of rivers. They are so soft that they cannot sustain, without settlement, even the load of a house, to say nothing of a heavier structure; nor can they be materially



REINFORCED CONCRETE WAREHOUSE BUILT FOR NELSON-MORRIS COMPANY, CHICAGO, ON SITE OF OLD BUILDING WHERE CHIEF HORAN AND A NUMBER OF FIREMEN LOST THEIR LIVES

This view shows distributing chutes and gives floor construction in various stages—form work, steel reinforcement, and finishing floor

until it reaches and is supported by a firm soil or by rock, which may be 50 or even 100 feet below the surface. The general methods of accomplishing these results will be detailed in the following pages.

Examination of Soil with Auger. The first step is to excavate the surface soil to the depth at which it would be convenient to place the foundation and at which the soil appears, from mere inspection, to be sufficiently firm for the purpose. An examination of the trenches or foundation pits with a post auger or steel bar will generally be sufficient to determine the nature of the soil for any ordinary building. The depth to which such an examination can be made with a post auger or steel bar will depend on the nature of the soil. In ordinary soils there will not be much difficulty in extending such an examination 3 to 6 feet below the bottom of the foundation pits. In common soils or clay, borings 40 feet deep, or even deeper, can readily be made with a common wood auger, turned by men. From the samples brought up by the auger, the nature of the soil can be determined; but nothing of the compactness of the soil can be determined in this manner.

Testing Compressive Value. In order to test a soil to find its compressive value, the bottom of the pit should be leveled for a considerable area, and stakes should be driven at short intervals in each direction. The elevations of the tops of all the stakes should be very accurately taken with a spirit level. For convenience, all stakes should be driven to the same level. A mast whose base has an area one foot square can support a platform which may be loaded with several tons of building material, such as stone, brick, steel, etc. This load can be balanced with sufficient closeness so that some very light guys will maintain the unstable equilibrium of the platform. As the load on the platform is greatly increased, at some stage it will be noted that the mast and platform have begun to sink slightly, and also that the soil in a circle around the base of the mast has begun to rise. This is indicated by the rising of the tops of the stakes. Even a very ordinary soil may require a load of five or six tons on a square foot before any yielding will be observable. One advantage of this method lies in the fact that the larger the area of the foundation, the greater will be the load per square foot which may

be safely carried, and that the uncertainty of the result is on the safe side. A soil which might yield under a load concentrated on a mast one foot square would probably be safe under that same unit load on a continuous footing which was perhaps three feet wide; and if, in addition, a factor of safety of three or four was used, there would probably be no question as to the safety. Such a test need be applied only to an earthy soil. It would be practically impossible to produce a yielding by such a method on any kind of rock or even on a compacted gravel.

Bearing Power of Ordinary Soils. A distinction must be maintained between the crushing strength of a cube of rock or soil, and the bearing power of that soil when it lies as a mass of indefinite extent under some structure. A soil can fail only by being actually displaced by the load above it, or because it has been undermined, perhaps by a stream of water. A sample of rock which might crush with comparative ease, when tested as a six-inch cube in a testing machine, will probably withstand as great a concentration of load as it is practicable to put upon it by any engineering structure. Even a gravel, which would have absolutely no strength if an attempt were made to place a cube of it in a testing machine, will be practically immovable when lying in a pit where it is confined laterally in all directions.

Rock. The ultimate crushing strength of stone varies greatly. The crushing strength is usually determined by making tests on small cubes. Tests made on prisms of a less height than width show a much greater strength than tests made on cubes of the same material, which shows that the bearing strength of rock on which foundations are built is much greater than the cubes of this stone. In Table I, Part I, the lowest value given for the crushing strength of a cube is 2,894 pounds per square inch, which is equal to 416,736 pounds per square foot. This shows that any ordinary stone which is well imbedded will carry any load of masonry placed on it.

Sand and Gravel. Sand and gravel are often found together. Gravel alone, when of sufficient thickness, makes one of the firmest and best foundations. Dry sand or wet sand, when prevented from spreading laterally, forms one of the best beds for foundations; but it must be well protected from running water, as it is easily moved

8,000 pounds per square foot; and when compact and well cemented, from 8,000 to 10,000 pounds per square foot. Ordinary gravel, well bedded, will safely bear a load of 6,000 to 8,000 pounds per square foot; and when well cemented, from 12,000 to 16,000 pounds per square foot.

Clay. There is great variation in clay soils, ranging from a very soft mass which will squeeze out in all directions when a very small pressure is applied, to shale or slate which will support a very heavy load. As the bearing capacity of ordinary clay is largely dependent upon its dryness it is, therefore, very important that a clay soil should be well drained, and that a foundation laid on such a soil should be at a sufficient depth to be unaffected by the weather. If the clay cannot be easily drained, means should be taken to prevent the penetration of water. When the strata are not horizontal, great care must be taken to prevent the flow of the soil under pressure. When gravel or coarse sand is mixed with the clay, the bearing capacity of the soil is greatly increased.

The bearing capacity of a soft clay is from 2,000 to 4,000 pounds per square foot; of a thick bed of medium dry clay, 4,000 to 8,000 pounds per square foot, and for a thick bed of dry clay, 8,000 to 10,000 pounds per square foot.

Soft or Semi-Liquid Soils. The soils of this class include mud, silt, quicksand, etc., and it is necessary to remove them entirely or to reach a more solid stratum under the softer soil; or, sometimes, the soil can be consolidated by adding sand, stone, etc. The manner of improving such a soil will be discussed later. In the same way that water will bear up a boat, a semi-liquid soil will support, by the upward pressure, a heavy structure. For a soil of this kind, it is very difficult to give a safe bearing value; perhaps 500 to 1,500 pounds per square foot is as much as can be supported without too great a settlement.

Improving a Compressible Soil. The general method of improving a compressible soil consists in making the soil more dense. This may be done by driving a large number of piles into the soil, especially if the piles will be always under the water line in that ground. Driving the piles compresses the soil; and if the piles are always under water, they will be free from decay. If the soil is sufficiently

form even temporarily, the pile may be drawn and then the hole immediately filled with sand, which is rammed into the hole as compactly as possible. This gives us a type of piling known as sand piles.

A soft, clayey subsoil may frequently be improved by covering it with gravel, which is rammed and pressed into the clay. Such a device is not very effective, but it may sometimes be sufficiently effective for its purpose.

A subsoil is often very soft because it is saturated with water which cannot readily escape. Frequently, a system of deep drainage, which will reduce the natural level of the ground water considerably below the desired depth of the bottom of the foundation, will transform the subsoil into a dry, firm soil which is amply strong for its purpose. Even when the subsoil is very soft, it will sustain a heavy load, provided that it can be confined. While excavating for the foundations of the tower of Trinity Church in New York City, a large pocket of quicksand was discovered directly under the proposed tower. Owing to the volume of the quicksand, it was found to be impracticable to drain it all out; but it was also discovered that the quicksand was confined within a pocket of firm soil. A thick layer of concrete was, therefore, laid across the top, which effectively sealed up the pocket of quicksand, and the result has been perfectly satisfactory.

PREPARING THE BED

Preparing the Bed on Rock. The preparation of a rock bed on which a foundation is to be placed is a simple matter compared with that required for some soils on which foundations are placed. The bed rock is prepared by cutting away the loose and decayed portions of the rock and making the plane on which the foundation is placed perpendicular to the direction of the pressure. If the rock bed is an inclined plane, a series of steps can be made for the support of the foundation. Any fissures in the rock should be filled with concrete.

Whenever it is necessary to start the foundation of a structure at different levels, great care is required to prevent a break in the joints at the stepping places. The precautions to be taken are that the mortar joints must be kept as thin as possible; the lower part of the

foundations should be laid in cement mortar; and the work should proceed slowly. By following these precautions, the settlement in the lower part will be reduced to a minimum. These precautions apply to foundations of all classes.

Preparing the Bed on Firm Earth. Under this heading is included hard clay, gravel, and clean, dry sand. The bed is prepared by digging a trench deep enough so that the bottom of the foundation is below the frost line, which is usually 3 to 6 feet below the surface. Some provision, similar to that shown in Fig. 38, should be made for drainage.

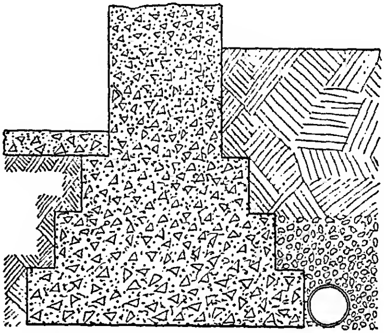


Fig. 38. Drainage for Foundation Wall

Care should be taken to proportion the load per unit of area so that the settlement of the foundation will be uniform.

Preparing the Bed on Wet Ground. The chief trouble in making an excavation in wet ground is in disposing of the water and preventing the wet soil from flowing into the excavation. In moderately wet soils, the area to be excavated is enclosed with sheet piling, as in Fig. 39. This piling usually consists of ordinary plank, 2 inches thick and 6 to 10 inches wide, and is often driven by hand; or it may be driven by methods that will be described later. The piling is driven in close contact, and in very wet soil it is necessary to drive a double row of the sheeting. To prevent the sheeting from being forced inwards, cross braces are used between the longitudinal timbers. When one

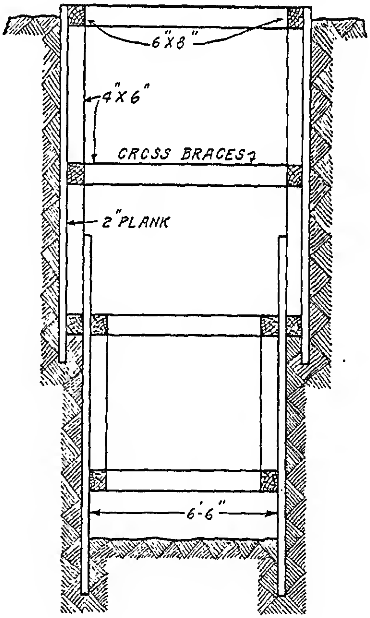


Fig. 39. Sheet Piling in Foundation Trenches

length of sheeting is not long enough, an additional length can be placed inside. A more extended discussion of pile driving will be given in the treatment of the subject "Piles".

The water can sometimes be bailed out, but it is generally necessary to use a hand or steam pump to free the excavation of water. Quicksand and very soft mud are often pumped out along with the water by a centrifugal or mud pump.

Sometimes, areas are excavated by draining the water into a hole the bottom of which is always kept lower than the general level of the bottom of the excavation. A pump may be used to dispose of the water drained into the hole or holes.

When a very soft soil extends to a depth of several feet, piles are usually driven at uniform distances over the area and a grillage is constructed on top of the piles. This method of constructing a foundation is discussed in the section on "Piles".

FOOTINGS

Requirements of Footing Course. The three requirements of a footing course are:

(1) That the area shall be such that the total load divided by the area shall not be greater than the allowable unit pressure on the subsoil.

(2) That the line of pressure of the wall, or pier, shall be directly over the center of gravity—and hence the center of upward pressure—of the base of the footings.

(3) That the footing shall have sufficient structural strength so that it can distribute the load uniformly over the subsoil.

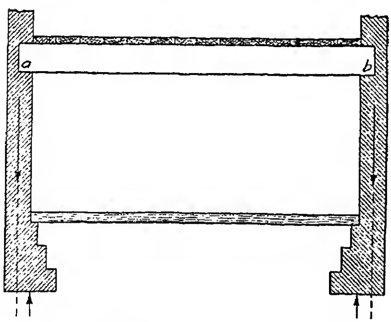


Fig. 40. Construction Where Lines of Downward and Upward Pressure on Footings Do Not Coincide

When it has been determined with sufficient accuracy how much pressure per square foot may be allowed on the subsoil (see pages 112, 113), and when the total load of the structure has been computed, it is a very simple matter to compute the width of continuous footings or the area of column footings.

The second requirement is very easily fulfilled when it is possible to spread the footings in all

figure will almost inevitably result in cracks in the building, unless some special device is adopted to prevent them. One general method is to introduce a tie of sufficient strength from *a* to *b*. The other general method is to introduce cantilever beams under the basement, which either extend clear across the building or else carry the load of interior columns so that the center of gravity of the combined loads will coincide with the central pressure line of the upward pressure of the footings.

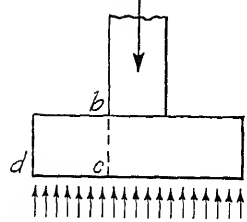


Fig. 41. Transverse Stresses in Footing Determining Its Thickness

The third requirement practically means that the thickness of the footing (bc, Fig. 41) shall be great enough so that the footing can resist the transverse stresses caused by the pressure of the subsoil on the area between c and d. When the thickness must be made very great, such as fh, Fig. 42, on account of the wide offset gh, material may be saved by cutting out the rectangle $ekml$. The thickness mo is computed for the offset go, just as in the first case; while the thickness km of the second layer may be computed from the offset kf. Where the footings are made of stone or of plain concrete, whose transverse strength is always low, the offsets are necessarily small; but when using timber, reinforced concrete, or steel I-beams, the offsets may be very wide in comparison with the depth of the footing.

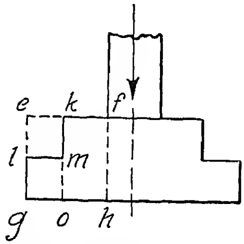


Fig. 42. Saving of Material in Very Thick Footing

Calculation of Footings. The method of calculation is to consider the offset of the footing as an inverted cantilever which is loaded with the calculated upward pressure of the subsoil against the footing. If Fig. 41 is turned upside down, the resemblance to the ordinary loaded cantilever will be more readily apparent. Considering a unit length l of the wall and the amount of the offset o equal to dc in Fig. 41, and calling P the unit pressure from

TABLE XVI

Ratio of Offset to Thickness for Footings of Various Kinds of Masonry

Kind of Masonry	Modulus of Rupture (Minimum and Maximum Values)	Average	Assumed Safe Value (R)	Pressure on Bottom of Footing (Tons per Square Foot)						
				0.5	1.0	1.5	2.0	2.5	3.0	3.5
Granite	1,200–1,365	1,280	130	2.5	1.8	1.45	1.25	1.1	1.0	0.95
Limestone	450– 900	675	70	1.8	1.3	1.05	0.9	0.8	0.75	0.7
Sandstone	135–1,100	525	55	1.6	1.15	0.95	0.8	0.75	0.65	0.6
Concrete (plain)										
1:2:4	400– 480	440	75	1.9	1.35	1.1	0.95	0.85	0.75	0.7
1:3:6	213– 246	230	40	1.4	1.0	0.8	0.7	0.6	0.55	0.5

the subsoil, we have Pol as the pressure on that area, and its lever arm about the point c is $\frac{1}{2}o$. Therefore, its moment equals $\frac{1}{2}Po^2l$. If t represents the thickness bc of the footing, the moment of resistance of that section equals $\frac{1}{6}Rlt^2$, in which R equals the unit compression (or unit tension) in the section. We therefore have the equation

$$\tfrac{1}{2}Po^2l=\tfrac{1}{6}Rlt^2$$

By transposition

$$\frac{o^2}{t^2}=\frac{R}{3P};\text{ or }\frac{o}{t}=\sqrt{\frac{R}{3P}} \qquad (2)$$

The fraction $\frac{o}{t}$ is the ratio of the offset to its thickness. The solution of the above equation, using what are considered to be conservatively safe values for R for various grades of stone and concrete, is given in Table XVI.

Example. The load on a wall has been computed as 19,000 pounds per running foot of the wall, which has a thickness of 18 inches just above the footing. What must be the breadth and thickness of granite slabs which may be used as a footing on soil which is estimated to bear safely a load of 2.0 tons per square foot?

Solution. Dividing the computed load (19,000) by the allowable unit pressure (2.0 tons equals 4,000 pounds), we have 4.75 feet as the required width of the footing.

$$\tfrac{1}{2}(4.75-1.5)=1.625 \text{ feet, the breadth of the offset } o$$

From the table we find that for a subsoil loading of 2.0 tons per square foot, the offset for granite may be 1.25 times its thickness. Therefore, $\frac{1.625}{1.25}=$

Although brick can be used as a footing course, the maximum possible offset, no matter how strong the brick may be, can only be a small part of the length of the brick—the brick being laid perpendicular to the wall. One requirement of a footing course is that the blocks shall be so large that they will equalize possible variations in the density and compressibility of the subsoil. This cannot be done by bricks or small stones. Their use should therefore be avoided in footing courses.

Beam Footings. Steel, and even wood, in the form of beams, are used to construct very wide offsets. This is possible on account of their greater transverse strength. The general method of calculation is identical with that given above, the only difference being that beams of definite transverse strength are so spaced that one beam can safely resist the moment developed in the footing in that length of wall. Wood can be used only when it will be always under water. Steel beams should always be surrounded by concrete for protection from corrosion.

Using Wood Beams. If we call the spacing of the beams s, the length of the offset o, and the unit pressure from the subsoil P, the moment acting on one beam equals $\frac{1}{2}Po^2s$. Calling w the width of the beam, t its thickness or depth, and R the maximum permissible fiber stress, the maximum permissible moment equals $\frac{1}{6}Rwt^2$. Placing these quantities equal, we have the equation

$$\frac{1}{2}Po^2s = \frac{1}{6}Rwt^2 \tag{3}$$

Having decided on the size of the beam, the required spacing may be determined.

Example. An 18-inch brick wall carrying a load of 12,000 pounds per running foot is to be placed on a soft, wet soil where the unit pressure cannot be relied on for more than 2,000 pounds per square foot. What must be the spacing of 10- by 12-inch footing timbers of long-leaf yellow pine?

Solution. The width of the footing is evidently $12{,}000 \div 2{,}000 = 6$ feet. The offset o equals $\frac{1}{2}(6 - 1.5) = 2.25$ feet $= 27$ inches. Since the unit of measurement for computing the transverse strength is the inch, the same unit must be employed throughout. Therefore, $P = \frac{2{,}000}{144}$; $R = 1{,}200$ pounds per

square inch; w = 10 inches; and t = 12 inches. Equation (3) may be rewritten

$$s = \frac{R w t^2}{3 P o^2}$$

Substituting the above values, we have

$$s = \frac{1{,}200 \times 10 \times 144}{3 \times \frac{2{,}000}{144} \times 729} = 56.9 \text{ in.}$$

This shows that the beams must be spaced 56.9 inches apart, center to center. These beams should be underlaid with thick planks, or even beams, laid close together, parallel with the wall, and for the entire width of the footing, for the double purpose of providing the full pressure area needed and also to tie the beams together. The span of the crossbeams is 56.9 inches or 4.74 feet. The clear space is 4.74−.83 or 3.91 feet. The working span is a little more than this, say even four feet. Then $M = (2000 \times 4) \times 48 \div 8$ or 48,000 inch-pounds. Placing this equal to $(R b h^2) \div 6$, in which $b = 12$, then $h = 4.48$ inches. Allowing a little for outside deterioration, the "planks" should be 5 inches thick.

Using Steel I-*Beams.* The method of calculation is the same as for wood beams, except that, since the strength of I-beams is more readily computable by reference to tables in the handbooks published by the manufacturers, such tables will be utilized. The tables always give the safe load which may be carried on an I-beam of given dimensions on any one of a series of spans varying by single feet. If we call W the total load (or upward pressure) to be resisted by a single cantilever beam, this will be one-fourth of the load which can safely be carried by a beam of the same size and on a span equal to the offset.

Example. Solve the previous example on the basis of using steel I-beams.

Solution. The offset is, necessarily, 2.25 feet; at 2,000 pounds per square foot, the pressure to be carried by the beams is 4,500 pounds for each foot of length of the wall. By reference to the tables and interpolating, a 6-inch I-beam weighing 12.25 pounds per linear foot will carry about 34,860 pounds on a 2-foot 3-inch span. One-fourth of this, or 8,715 pounds, is the load carried by a cantilever of that length. Therefore, 8,715 ÷ 4,500 = 1.936 feet = 23.25 inches, is the required spacing of such beams.

When comparing the cost of this method with the cost of others, the cost of the masonry-concrete filling must not be overlooked. A

steel should be immediately placed at the proper spacing; then the spaces between the beams should be filled in with concrete, care being taken to ram the concrete so thoroughly as to prevent voids. The concrete should extend up to a level at least two inches above the beams so as to protect the steel from rusting. In this case the spacing is 23.25 inches, and the net clear space about 20 inches. Since the concrete will be deeper than this, we may say, without numerical calculation, that the arching action of the concrete between the beams would be ample to withstand the soil pressure. The spacing of the beams should be neither so wide as to preclude safe arching action—which is unlikely—nor so narrow as to hinder thorough tamping of the concrete between them.

Design of Pier Footings. The above designs for footings have been confined solely to the simplest case of the footing required for a continuous wall. A column or pier must be supported by a footing which is offset from the column in all four directions. It is usually made square. The area is very readily obtained by dividing the total load by the allowable pressure per square foot on the soil. The quotient is the required number of square feet in the area of the footing. If a square footing is permissible—and usually it is preferable—the square root of that number gives the length of one side of the footing. Special circumstances frequently require a rectangular footing or even one of special shape. The problem of so designing a footing that the center of pressure of the load on a column shall be vertical over the center of pressure of the subsoil is solved in detail under "Column Footing", Part III, page 252. A column placed at the corner of a building which is located at the extreme corner of the property, and which cannot extend over the property line, must usually be supported by a compound footing. The principles involved are discussed in detail, under "Compound Footings", Part III, page 256.

The determination of the thickness of a footing depends somewhat upon the method used. When the grillage is constructed of I-beams, as illustrated in Fig. 43, the required strength of each series of beams is readily computed from the offset of each layer. If the footing consists of a single block of stone or a plate of concrete, either plain or reinforced, the thickness must be computed on the

tically, concentrated in the center. This problem is taken up under "Simple Footings", Part III, page 249.

Example. A column with a base 3 feet 4 inches square, carrying a total load of 630,000 pounds, is to be supported on a soil on which the permissible

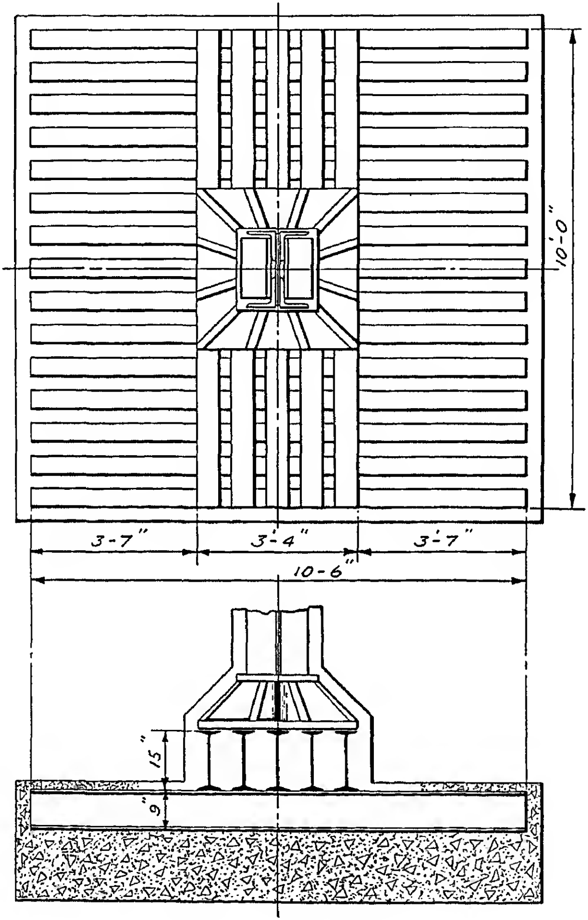


Fig. 43. Grillage of I-Beams

loading is estimated as three tons per square foot; an I-beam footing is to be used. Required, the design of the I-beams.

Solution. The required area of the footing is evidently 630,000 ÷ 6,000 = 105 square feet. Using a footing similar to that illustrated in Fig. 43, we shall make the lower layer of the footing, say 10 feet 6 inches by 10 feet wide. The length of the beams being 126 inches, and the column base being 40 inches

square, the offset from the column is 43 inches, or 3.58 feet on each side. Looking at a table of standard I-beams, we find that a 9-inch beam weighing 21 pounds per linear foot will carry 50,320 pounds on a span of four feet. For a span of 3.58 feet, the allowable load is $\frac{4.00}{3.58} \times 50{,}320$, or 56,220 pounds. Taking one-fourth of this, as in the example on page 120, we have 14,055 pounds which may be carried by each beam acting as a cantilever. The upward pressure of an offset 3.58 feet long and 10 feet wide, at the rate of 6,000 pounds per square foot, would be 214,800 pounds. Therefore, 15 such beams, each 10 feet 6 inches long, would be required in the lower layer. The upper layer must consist of beams 10 feet long on which the offset from the pier is 40 inches on each side. The group of beams under each of these upper offsets carries an upward pressure of 6,000 pounds per square foot on an area of 10 feet 6 inches by 3 feet 4 inches; total pressure, 210,000 pounds. The total load on each foot of width of the upper layer is 63,000 pounds. Looking at the tables, a 15-inch I-beam weighing 42 pounds per foot can carry a load, on a 10-foot span, of 62,830 pounds. The permissible load on a cantilever of this length would be one-fourth of this, or 15,700 pounds. The permissible load on a cantilever 3 feet 4 inches long will be in the ratio of 10 feet to 3 feet 4 inches, or, in this case, exactly three times as much, which would be 47,100 pounds. The total of 210,000 pounds, divided by 47,100, will show that although five such beams will have a somewhat excessive strength, four would not be sufficient; therefore five beams should be used. The lower layer of beams have a flange width of 4.33 inches each. The 15 beams, distributed over a space of 10 feet, or 120 inches, would be about 8 inches apart, leaving 3.67 inches net space between them, which is sufficient for ramming the concrete. The five upper beams each have a flange width of 5.5 inches, which would use up 27.5 inches of the 40 inches width of the column base, leaving 12.5 inches for the four spaces, or $3\frac{1}{8}$ inches per space, which is again sufficient, although it is about as close as is desirable. It should not be forgotten that surrounding all these beams in both layers with concrete adds somewhat to the strength of the whole footing, but that no allowance is made for this additional strength in computing dimensions. It merely adds an indefinite amount to the factor of safety.

PILE FOUNDATIONS

Piles. The term pile is generally understood to be a stick of timber driven in the ground to support a structure. This stick of timber is generally thought of as the body of a small tree; but timber in many shapes is used for piling. Sheet piling, for example, is generally much wider than thick. Cast iron and wrought iron have also been used for all forms of piling. Structural steel has also been used for this purpose. Within the last few years, concrete and reinforced concrete piles have been used quite extensively in place of wood or cast-iron piles.

Cast-Iron Piles. Cast-iron piles have been used to some extent. The advantages claimed for these piles are that they are not subject to decay; they are more readily driven than wood piles in stiff clays or stony ground; and they have a greater crushing strength than wood piles. The latter quality will apply only when the pile acts as a column. The greatest objection to these piles is that they are deficient in transverse strength to resist sudden blows. This objection applies only in handling them before they are driven, and to those which, after being driven, are exposed to blows from ice and logs, etc. When driving cast-iron piles, a block of wood is placed on top of the pile to receive the blow; and, after being driven, a cap with a socket in its lower side is placed upon the pile to receive the load. Generally, lugs or flanges are cast on the sides of the piles, to which bracing may be attached for fastening them in place.

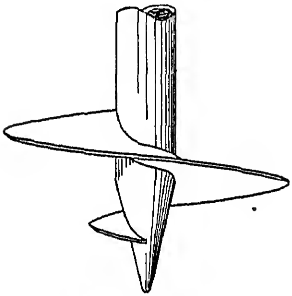


Fig. 44. Screw Pile

Screw Piles. This term refers to a type of metal pile whose use is limited, but which is apparently very effective where it has been used. It consists essentially of a steel shaft, 3 to 8 inches in diameter, strong enough to act as a column, and also to withstand the twisting to which it is subjected while the pile is being sunk, Fig. 44. At the lower end of the shaft is a helicoidal surface having a diameter of perhaps five feet. Such piles can be used only in comparatively soft soil, and their use is practically confined to foundations in sandbanks on the shore of the ocean. To sink such piles, they are screwed into place by turning the vertical shaft with a long lever. Such a sinking is usually assisted by a water jet, which will be described later.

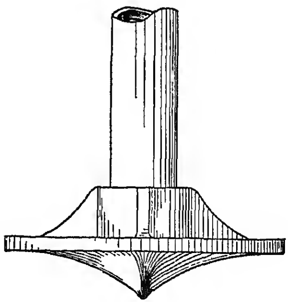


Fig. 45. Disk Pile

Disk Piles. A variation of the screw pile is the disk pile, Fig. 45, which, as its name implies, has a circular disk in place of a heli-

conical surface. Such a pile can be sunk only by use of a water jet, the pile being heavily loaded so that it shall be forced down.

Wood Bearing Piles. Specifications for wood piles generally require that they shall have a diameter of from 7 to 10 inches at the smaller end, and 12 to 15 inches at the larger end. Older specifications were quite rigid in insisting that the tree trunks should be straight, and that the piles should be free from various kinds of minor defects; but the growing scarcity of timber is modifying the rigidity of these specifications, provided the most essential qualifications of strength and durability are provided for. Timber piles should have the bark removed before being driven, unless the piles are to be always under water. They should be cut square at the driving end, and pointed at the lower end. When they are to be driven in hard, gravelly soil, it is often specified that they shall be shod with some form of iron shoe. This may be done by means of two straps of wrought iron, which are bent over the point so as to form four bands radiating from the point of the pile, Fig. 46. By means of holes through them, these bands are spiked to the piles. Another method, although it is considered less effective on account of its liability to be displaced during driving, is to use a cast-iron shoe. These shoes are illustrated in Fig. 47. It is sometimes specified that piles shall be driven with the butt end, or larger end, down, but there seems to be little if any justification for such a specification. The resistance to driving is considerably greater, while their ultimate bearing power is but little if any greater. If the driving of piles is considered from the standpoint of compacting the soil, as already discussed on page 113, then driving

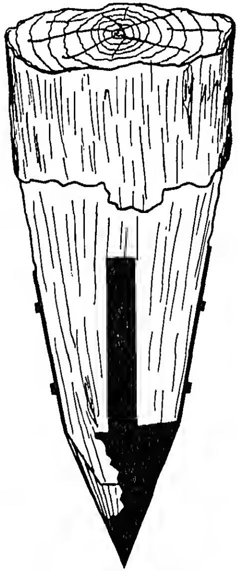


Fig. 46. Wrought-Iron Pile-Shoe

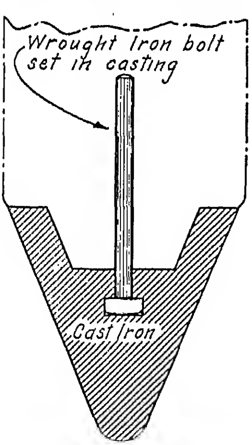


Fig. 47. Cast-Iron Pile-Shoe

the piles with the small end down will compact the soil more effectively than driving them butt end down.

White pine, spruce, or even hemlock may be used in soft soils; yellow pine in firmer ones; and oak, elm, beech, etc., in the more compact soils. The piles are usually driven from $2\frac{1}{2}$ to 4 feet apart each way, center to center, depending on the character of the soil and the load to be supported. Timber piles, when partly above and partly under water, will decay very rapidly at the water line.

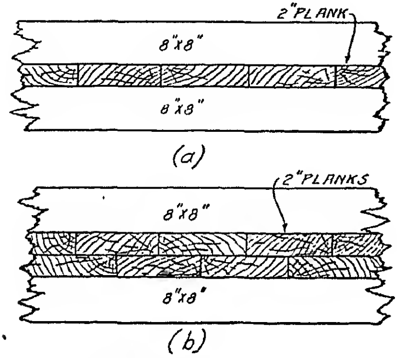


Fig. 48. Single and Double Sheet Piling

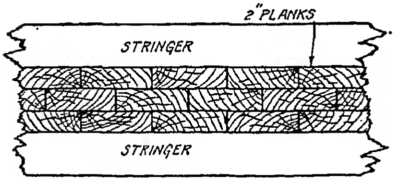


Fig. 49. Triple Sheet Piling for Cofferdams

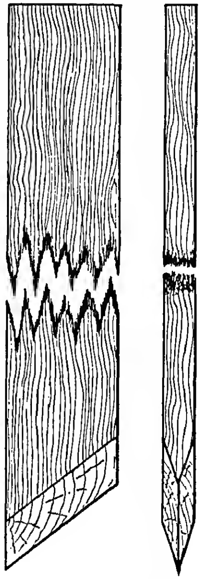


Fig. 50. Bevel Point for Sheet Pile

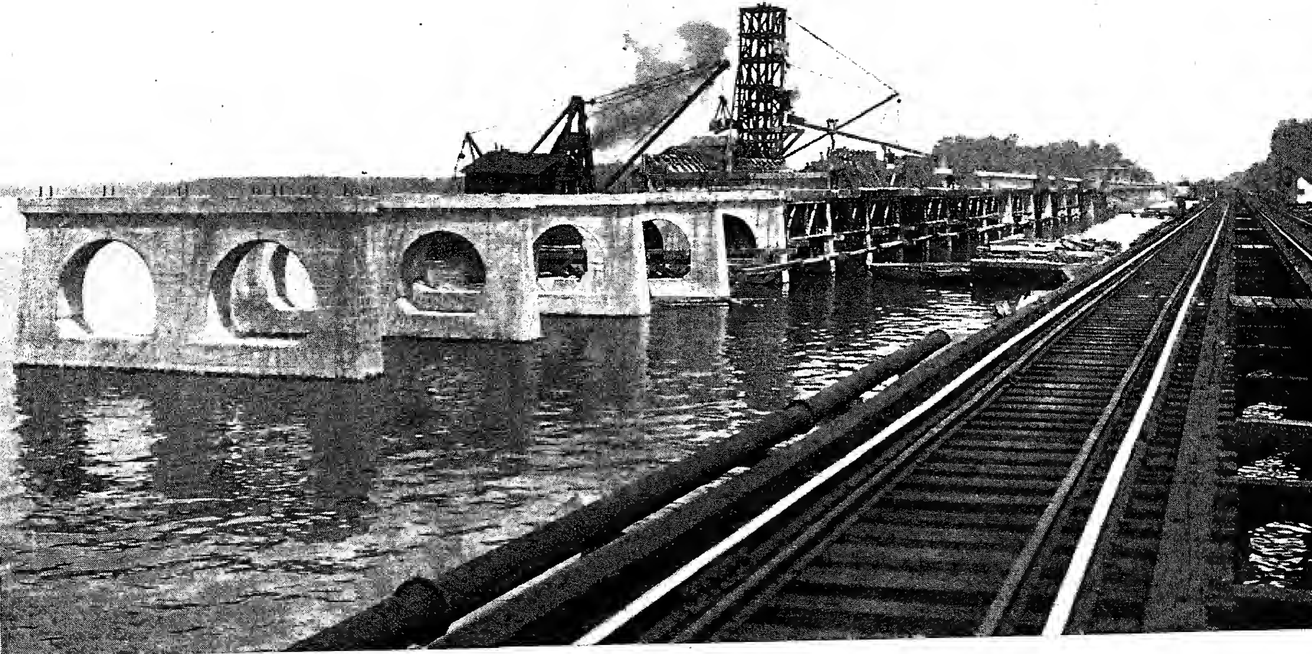
This is owing to the alternation of dryness and wetness. In tidal waters they are destroyed by the marine worm known as the *teredo*.

The American Railway Engineering Association recommends the following specifications for piling:

Piles shall be cut from sound, live trees; shall be close-grained and solid; free from defects such as injurious ring shakes, large and unsound knots, decay, or other defects that will materially impair their strength. The taper from butt to top shall be uniform and free from short bends.

All piles except foundation piles shall be peeled.

Sheet Piling. Ordinary planks, 2 or 3 inches thick, and wider than they are thick, are, when driven close together, known as *sheet*



SOLID CONCRETE SLAB BRIDGE OVER BUSH RIVER ON MAIN LINE OF PENNSYLVANIA RAILROAD BETWEEN PHILADELPHIA AND WASHINGTON

Photo by Boston Photo News Company, Boston, Massachusetts

reduced by using a second row of plank, breaking joints with the first row, as shown in Fig. 48-b. When it is required that the sheet piling shall form a water-tight wall, such as a cofferdam, three thicknesses of plank are generally used with joints as arranged in Fig. 49.

Sheet piling is usually driven in close contact, either to prevent leakage, to confine puddle in cofferdams, to prevent the materials of a foundation from spreading, or to guard a foundation from being

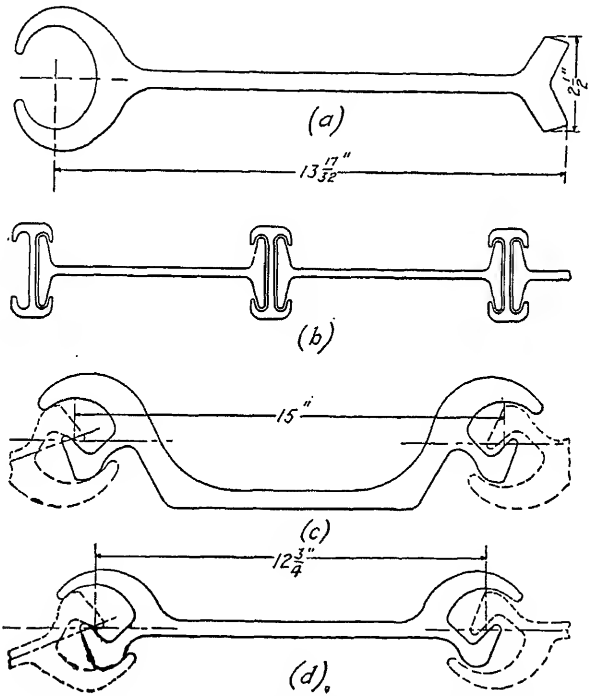


Fig. 51. Types of Sheet Steel Piling. (a) Carnegie Steel Company; (b) Jones and Laughlin; (c) Lackawanna Steel Company (Arched Web Section); (d) Lackawanna Steel Company (Straight Web Section).

undermined by a stream of water. To make wood piles drive with their sides close against each other, they are cut obliquely at the bottom, as shown in Fig. 50. They are kept in place, while being driven, by means of two longitudinal stringers or *wales*. These wales are supported by gage piles previously driven, which are several feet apart.

ave developed the manufacture of *steel sheet piling,* which can be e-drawn and used many times. The forms of steel for sheet piling are early all patented. The cross sections of a few of them are shown Fig. 51. One feature of some of the designs is the possible flexibility secured in the outline of the dam without interfering with the water-tightness.

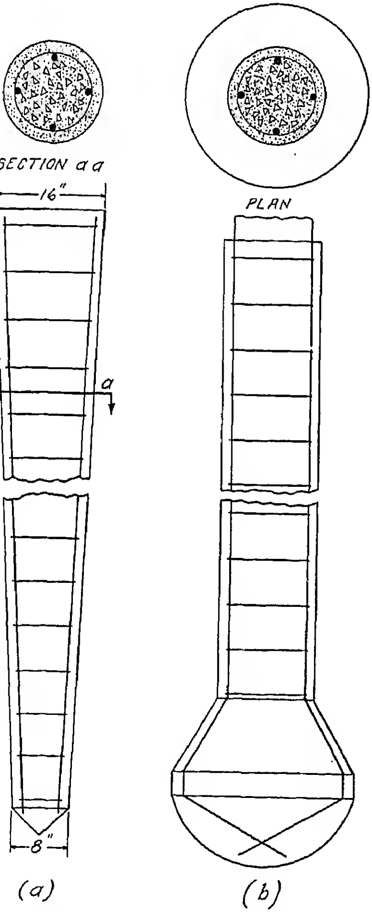


Fig. 52. Reinforced-Concrete Piles

Concrete and Reinforced-Concrete Piles. Concrete and reinforced-concrete piles may be classified under two headings: (*a*) those where the piles are formed, hardened, and driven very much the same as any pile is driven; (*b*) those where a hole is made in the ground, into which concrete is rammed and left to harden.

Reinforced-concrete piles which have been formed on the ground are designed as columns with vertical reinforcement connected at intervals with horizontal bands. These piles are usually made round or octagonal in section, and a steel or cast-iron point is used.

Fig. 52-a shows a type of pile that is commonly used when constructed in forms, hardened, and driven the same as a wood pile. These piles must be reinforced with steel so that they can be handled.

Fig. 52-b shows the general plan of a type of pile that has been ed to some extent along the seashore where piles can be jetted. ey are usually molded in a vertical position and as soon as they be handled are jetted in place. These piles are not dependent

convey the load direct to the sand under the enlarged end. Piles of this type have been used for loads of 50 to 60 tons. They cannot be used in clusters but each pile must be of sufficient size to support the entire load at any given point.

Raymond Concrete Pile. The Raymond concrete pile, Fig. 53, is constructed in place. A collapsible steel pile core is encased in a thin, closely-fitting, sheet-steel shell. The core and shell are driven to the required depth by means of a pile driver. The core is so constructed that when the driving is finished, it is collapsed and withdrawn, leaving the shell in the ground, which acts as a mold for the concrete. When the core is withdrawn, the shell is filled with concrete, which is tamped during the filling process. These piles are usually 18 to 20 inches in diameter at the top, and 6 to 8 inches at the point. When it is desirable, the pile can be made larger at the small end. The sheet steel used for these piles is usually No. 20 gage. When it is desirable to reinforce these piles, the bars are inserted in the shell after the core has been withdrawn and before the concrete is placed.

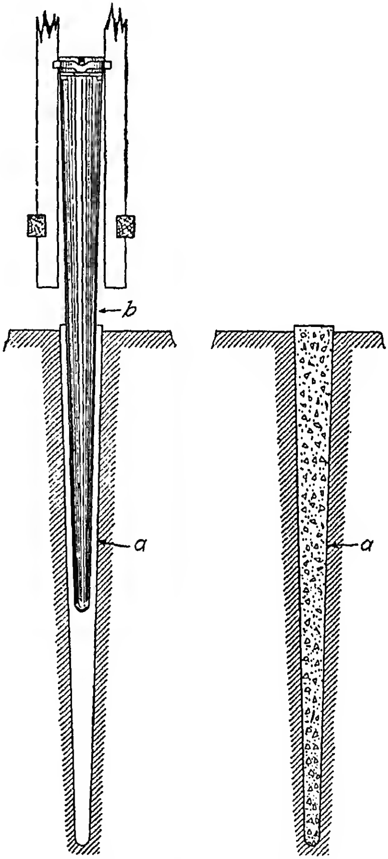


Fig. 53. Raymond Concrete Pile

Simplex Concrete Pile. The different methods for producing the Simplex pile cover the two general classifications of concrete piles—namely, those molded in place, and those molded above ground and driven with a pile driver. Fig. 54 shows the standard methods of producing the Simplex pile; *A* shows a cast-iron point

alligator-point driving form. The only difference between the two forms shown in this figure is that the alligator point is withdrawn and the cast-iron point remains in the ground. The concrete in either type is compacted by its own weight. As the form is removed, the concrete comes in contact with the soil and is bonded with it. A danger in using this type of pile is that, if a stream of water is

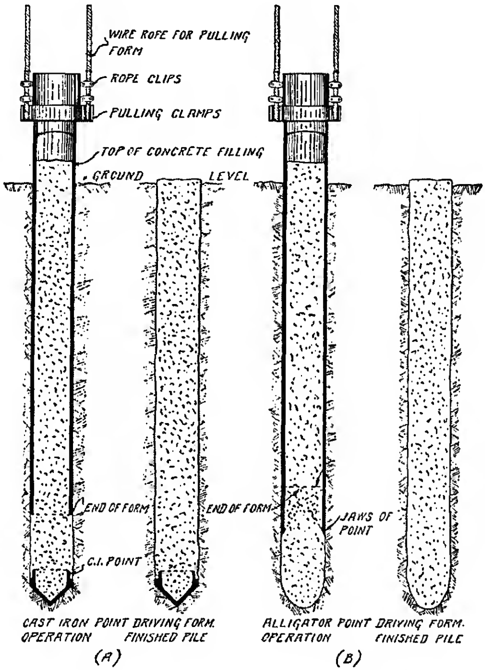


Fig. 54. Standard Simplex Concrete Piles

encountered, the cement may be washed out of the concrete before it has a chance to set.

A shell pile and a molded and driven pile are also produced by the same company which manufactures the Simplex, and are recommended for use under certain conditions. Any of these types of piles can be reinforced with steel. This company has driven piles 20 inches in diameter and 75 feet long.

Steel-Shelled Concrete Piles. In excavating for the foundation of a 16-story building at 14th Street and 5th Avenue, New York, a

below the bottom of the general excavation. A wall column of the building to be constructed was located at this point, with its center only 15 inches from the party line. The estimated load to be supported by this column was about 500 tons. It was finally decided to adopt steel piles which would afford the required carrying capacity in a small, compact cluster, and would transfer the load, as well as the other foundations, to the solid rock. These piles, 5 in number, were driven very close to an existing wall and without endangering it. Each pile was about 15 feet long, and was made with an outer shell consisting of a steel pipe, $\frac{3}{8}$ inch thick and 12 inches inside diameter, filled with Portland-cement concrete, reinforced with four vertical steel bars, 2 inches in diameter. This gave a total cross-sectional area of 27.2 square inches of steel, with an allowed load of 6,000 pounds per square inch, and 100.5 square inches of concrete on which a unit stress of 500 pounds was allowed. This utilizes the bearing strength of the external shell, and enables the concrete filling to be loaded to the maximum permitted by the New York Building Laws. The tubes and bars have an even bearing on hard bed rock, to which the former were sunk by the use of a special air hammer and an inside hydraulic jet. The upper ends of the steel tubes and reinforcing bars were cut off after the piles were driven. The work was done with care, and a direct contact was secured between them and the finished lower surfaces of the cast-iron caps, without the intervention of steel shims.*

Cushing Pile Foundation. A combination of steel, concrete, and wood piles is known as the Cushing pile foundation. A cluster of piles is driven so that it may be surrounded by a wrought-iron or steel cylinder, which is placed over them, and which is sunk into the soil until it is below any chance of scouring action on the part of any current of water. The space between the piles and the cylinder is then surrounded with concrete. Although the piles are subject to decay above the water line, yet they are so thoroughly surrounded with concrete that the decay is probably very slow. The steel outer casing is likewise subject to deterioration, but the strength of the whole combination is but little dependent on the steel. If such foundations are sunk at the ends of the two trusses of a bridge, and are suitably cross-braced, they form a very inexpensive and yet

effective pier for the end of a truss bridge of moderate span. The end of such a bridge can be connected with the shore bank by means of light girders, and by this means the cost of a comparatively expensive masonry abutment may be avoided.

CONSTRUCTION FACTORS

Bearing Power of Piles. Pile foundations act in a variable combination of two methods of support. In one case the piles are driven into the soil to such a depth that the frictional resistance of the soil to further penetration of the pile is greater than any load which will be placed on the pile. As the soil becomes more and more soft, the frictional resistance furnished by the soil is less and less; and it then becomes necessary that the pile shall penetrate to a stratum of much greater density, into which it will penetrate but little if any. Under such conditions, the structure rests on a series of columns (the piles) which are supported by the hard subsoil, and whose action as columns is very greatly assisted by the density of the very soft soil through which the piles have passed. It practically makes but little difference which of these methods of support exists in any particular case. The piles are driven until the resistance furnished by each pile is as high as is desired. The resistance against the sinking of a pile depends on such a very large variety of conditions, that attempts to develop a formula for that resistance, based on a theoretical computation taking in all these various factors, are practically useless. There are so many elements of the total resistance, which are so large and also so very uncertain, that they entirely overshadow the few elements which can be precisely calculated. Most formulas for pile driving are based on the general proposition that the resistance of the pile multiplied by the distance which it moves during the last blow equals the weight of the hammer multiplied by the distance through which it falls. Expressing this algebraically, we have

$$Rs = wh$$

where R is resistance of pile; s is penetration of pile during last blow; w is weight of hammer; and h is height of fall of hammer.

Practically, such a formula is considerably modified, owing to the fact that the resistance of a pile, or its penetration for any blow,

depends considerably on the time which has elapsed since the previous blow. This practically means that it is far easier to drive the pile when the blows are delivered very rapidly; and also that when a pause is made in the driving, for a few minutes or for an hour, the penetration is very much less (and the apparent resistance very much greater), on account of the earth settling around the pile during the interval. The most commonly used formula for pile driving is known as the *Engineering News formula,* which, when used for ordinary hammer-driving, is

$$R = \frac{2\,w\,h}{s+1} \tag{4}$$

This formula is fundamentally the same as the above formula, except that R is safe load, in pounds; s is penetration per blow in inches; w is weight of hammer in pounds; and h is height of fall of hammer in feet.

In the above equation, w is considered a free-falling hammer (not retarded by hammer rope) striking a pile having a sound head. If a friction-clutch driver is used, so that the hammer is retarded by the rope attached to it, the penetration of the pile is commonly assumed to be just one-half what it would have been had no rope been attached, that is, had it been free-falling.

Also, the quantity s is arbitrarily increased by 1, to allow for the influence of the settling of the earth during ordinary hammer pile driving, and a factor of safety of 6 for safe load has been used. In spite of the extreme simplicity of this formula compared with that of others which have attempted to allow for all possible modifying causes, this formula has been found to give very good results. When computing the bearing power of a pile, the penetration of the pile during the last blow is determined by averaging the total penetration during the last five blows.

The pile-driving specifications adopted by the American Railway Engineering Association, require that:

All piles shall be driven to a firm bearing satisfactory to the Engineer, or until five blows of a hammer weighing 3,000 pounds, falling 15 feet (or a hammer and fall producing the same mechanical effect), are required to drive a pile one-half ($\frac{1}{2}$) inch per blow, except in soft bottom, when special instruc-

This is equivalent to saying (applying the *Engineering News formula*) that the piles must have a bearing power of 60,000 pounds.

Examples. 1. The total penetration during the last five blows was 14 inches for a pile driven with a 3,000-pound hammer. During these blows the average drop of the hammer was 24 feet. How much is the safe load?

$$\frac{2\,w\,h}{s+1}=\frac{2\times 3{,}000\times 24}{(\frac{1}{5}\times 14)+1}=\frac{144{,}000}{3.8}=37{,}895 \text{ pounds}$$

2. It is required, if possible, to drive piles with a 3,000-pound hammer until the indicated resistance is 70,000 pounds. What should be the average penetration during the last five blows when the fall is 25 feet?

$$70{,}000=\frac{2\,w\,h}{s+1}=\frac{2\times 3{,}000\times 25}{s+1}=\frac{150{,}000}{s+1}$$

$$s=\frac{150{,}000}{70{,}000}-1=2.14-1=1.14 \text{ inches}$$

The last problem suggests a possible impracticability, for it may readily happen that when the pile has been driven to its full length its indicated resistance is still far less than that desired. In some cases, such piles would merely be left as they are, and additional piles would be driven beside them, in the endeavor to obtain as much total resistance over the whole foundation as is desired.

The above formula applies only to the drop-hammer method of driving piles, in which a weight of 2,500 to 3,000 pounds is raised and dropped on the pile.

When the steam pile driver is used, the blows are very rapid, about 55 to 65 per minute. On account of this rapidity the soil does not have time to settle between the successive blows, and the penetration of the pile is much more rapid, while of course the resistance after the driving is finished is just as great as is secured by any other method. On this account, the above formula is modified so that the arbitrary quantity added to s is changed from one to 0.1, and the formula becomes

$$R=\frac{2\,w\,h}{s+0.1} \qquad (5)$$

Methods of Driving Piles. There are three general methods of driving piles—namely, by using (1) a falling weight; (2) the erosive action of a water jet; or (3) the force of an explosive. The third method is not often employed, and will not be further discussed. In constructing foundations for small highway bridges, well-augers

rammed around them.

Drop-Hammer Pile Driver. This method of driving piles consists in raising a hammer made of cast iron, and weighing 2,500 to 3,000 pounds, to a height of 10 to 30 feet, and then allowing it to fall freely on the head of the pile. The weight is hoisted by means of a hoisting engine, or sometimes by horses. When an engine is used for the hoisting, the winding drum is sometimes merely released, and the weight in falling drags the rope and turns the hoisting drum as it falls. This reduces the effectiveness of the blow, and lowers the value of s in the formula given, as already mentioned. To guide the hammer in falling, a frame, consisting of two uprights called leaders, about 2 feet apart, is erected. The uprights are usually wood beams, and are from 10 to 60 feet long. Such a simple method of pile driving, however, has the disadvantage, not only that the blows are infrequent—not more than 20 or even 10 per minute—but also that the effectiveness of the blows is reduced on account of the settling of the earth around the piles between the successive blows. On this account, a form of pile driver known as the steam pile driver is much more effective and economical, even though the initial cost is considerably greater.

Steam-Hammer Pile Driver. The steam pile driver is essentially a hammer which is attached directly to a piston in a steam cylinder. The hammer weighs about 4,000 pounds, is raised by steam to the full height of the cylinder, which is about 40 inches, and is then allowed to fall freely. Although the height of fall is far less than that of the ordinary pile driver, the weight of the hammer is about double, and the blows are very rapid (about 50 to 65 per minute). As before stated, on account of this rapidity, the soil does not have time to settle between blows, and the penetration of the pile is much more rapid, while, of course, the ultimate resistance, after the driving is finished, is just as great as that secured by any other method.

Driving Piles with Water Jet. When piles are driven in a situation where a sufficient supply of water is available, their resistance during driving may be very materially reduced by attaching a pipe to the side of the pile and forcing water through the pipe by means of a pump. The water softens and scours out the soil immediately

underneath the pile, and enables the pile to settle in the hole easily. The water returns to the surface along the sides of the pile and assists in reducing the frictional resistance. In very soft soils, and in sand, piles may thus be jetted by merely weighting them with a few hundred pounds while the force pump is in action. When the pile is practically down to the depth to which it is to be jetted, it should be struck a few blows with a light hammer to settle it firmly in the bottom of the hole. Of course, the only method of testing such resistance of the pile is by actually loading a considerable weight upon it. This method of using a water jet is chiefly applicable in structures which are on the banks of streams or large bodies of water.

Splicing Piles. On account of the comparatively slight resistance offered by piles in swampy places, it sometimes becomes necessary to splice two piles together. The splice is often made by cutting the ends of the piles perfectly square so as to make a good butt joint. A hole 2 inches in diameter and 12 inches deep is bored in each of the butting ends, and a dowel pin 23 inches long is driven in the hole bored in the first pile; the second pile is then fitted on the first one. The sides of the piles are then flattened, and four 2- by 4-inch planks, 4 to 6 feet long, are securely spiked on the flattened sides of the piles. Such a joint is weak at its best, and the power of lateral resistance of a joint pile is less than would be expected from a single stick of equal length. Nevertheless, such an arrangement is in some cases the only solution.

Pile Caps. One practical trouble in driving piles, especially those made of soft wood, is that the end of the pile will become crushed or broomed by the action of the heavy hammer. Unless this crushed material is trimmed off the head of the pile, the effect of the hammer is largely lost in striking this cushioned head. This crushed portion of the top of a pile should always be cut off just before the test blows are made to determine the resistance of the pile, since the resistance of a pile indicated by blows upon it, if its end is broomed, will apparently be far greater than the actual resistance of the pile.

The steam pile driver does not produce such an amount of brooming as is caused by the ordinary pile driver and this is another

head of the pile, it shows either that the fall is too great or that the pile has already been driven to its limit. Whenever the pile refuses to penetrate appreciably for each blow, it is useless to drive it any farther, since added blows can only have the effect of crushing the pile and rendering it useless. It has frequently been discovered that piles which have been hammered after they had been driven to their limit have become broken and crushed, perhaps several feet underground. In such cases, their supporting power is very much reduced.

Usually about two inches of the head is chamfered off to prevent this bruising and splitting in driving the pile. A steel band, 2 to 3 inches wide and $\frac{1}{2}$ to 1 inch thick, is often hooped over the head of the pile to assist in keeping it from splitting. These devices have led to the use of a cast-iron cap for the protection of the head of the pile. The cap is made with two tapering recesses, one to fit on the chamfered head of the pile, while in the other is placed a piece of hard wood, on which the hammer falls.

Sawing Off the Piles. When the piles have been driven, they are sawed off to bring the tops of them to the same elevation, that they may have an even bearing surface. When the tops of the piles are above water, this sawing is usually done by hand; and when under water, by machinery. The usual method of cutting piles off under water is by means of a circular saw on a vertical shaft, which is supported on a special frame, the saw being operated by the engine used in driving the piles.

Finishing the Foundations. A pile supports a load coming on an area of the foundation which is approximately proportional to the spacing between the piles. This area, of course, is several times the area of the top of the pile. It is therefore necessary to cap at least a group of the piles with a platform or grillage which not only will support any portion of the load located between the piles, but which also will tend to prevent a concentration of load on one pile. When the heads of the piles are above water, a layer of concrete is usually placed over them, the concrete resting on the ground between

The piles are thus firmly anchored together at their tops. When reinforced-concrete structures are supported on piles or other concentrated points of support, the heads of the piles are usually connected by reinforced-concrete beams, which will be described in Part III. Sometimes a platform of heavy timbers is constructed on top of the piles, to assist in distributing the load; and in this case the concrete is placed on the platform, as shown in Fig. 56.

When the heads of the piles are under water, it is always necessary to construct a grillage of heavy timber and float it into place, unless a cofferdam is constructed and the water pumped out, in which case the foundation can be completed as already described. The timbers used to cap the piles in constructing a grillage are usually

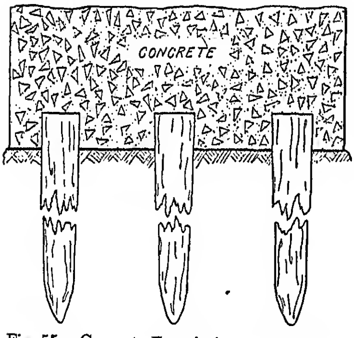


Fig. 55. Concrete Foundation on Wood Piles

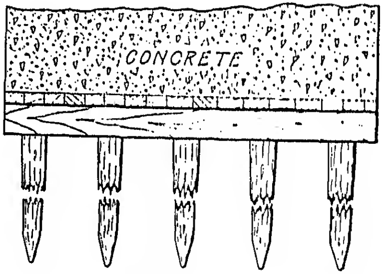


Fig. 56. Timber Foundation on Wood Piles

about 12 by 12 inches, and are fastened to the head of each pile by a driftbolt—a plain bar of steel. A hole is bored in the cap and into the head of the pile, in which the driftbolt is driven. The section of the driftbolt is always larger than the hole into which it is to be driven; that is, if a 1-inch round driftbolt is to be used, a $\frac{7}{8}$-inch auger would be used to bore the hole. The transverse timbers of the grillage are driftbolted to the caps.

Advantage of Concrete and Reinforced-Concrete Piles. A reinforced-concrete pile foundation does not materially differ in construction from a timber pile foundation. The piles are driven and capped, in the usual manner, with concrete ready for the superstructure. In comparing this type of piles with timber piles, they

and the disadvantage of being more expensive in first cost. Sometimes their use will effect a saving in the total cost of the foundation by obviating the necessity of cutting the piles off below the water line. The depth of the excavation and the volume of masonry may be greatly reduced, as shown in Fig. 57. In this figure is shown a comparison of the relative amount of excavation which would be necessary, and also of the concrete which would be required for the piles, thus indicating the economy which is possible in the items of excavation and concrete. There is also shown a possible economy in the number of piles required, since concrete piles can readily be

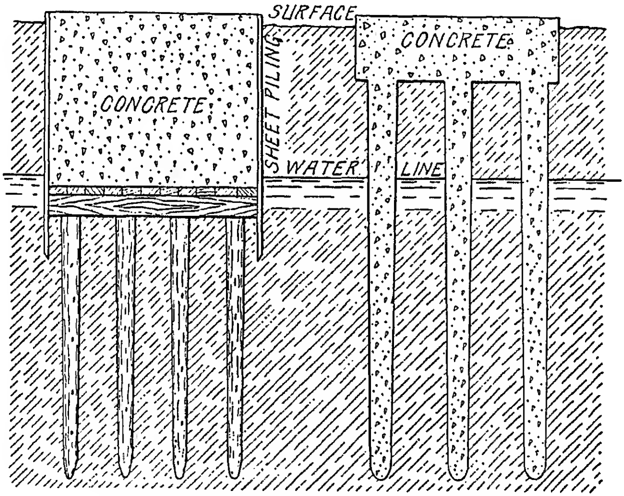


Fig. 57. Comparison of Wood and Concrete Piles

made of any desired diameter, while there is a practical limitation to the diameter of wood piles. Therefore a less number of concrete piles will furnish the same resistance as a larger number of wood piles. In Fig. 57 it is assumed that the three concrete piles not only take the place of the four wood piles in the width of the foundation, but there will also be a corresponding reduction in the number of piles in a direction perpendicular to the section shown. The extent of these advantages depends very greatly on the level of the groundwater line. When this level is considerably below the surface of the ground, the excavation and the amount of concrete required, in

always below the water line, will be correspondingly great, and the possible economy of concrete piles will also be correspondingly great.

The pile and cap being of the same material, they readily bond together and form a monolithic structure. The capping should be thoroughly reinforced with steel. Reinforced-concrete piles can be driven in almost any soil that a timber pile can penetrate, and they are driven in the same manner as the timber piles. A combination of the hammer and water jet has been found to be the most successful manner of driving them. The hammer should be heavy and drop a short distance with rapid blows, rather than a light hammer dropping a greater distance. For protection while being driven, a hollow cast-iron cap filled with sand is placed on the head of the pile. The cap shown in Fig. 58 has been used successfully in driving concrete piles. A hammer weighing 2,500 pounds was dropped 25 feet, 20 to 30 times per minute, without injury to the head.

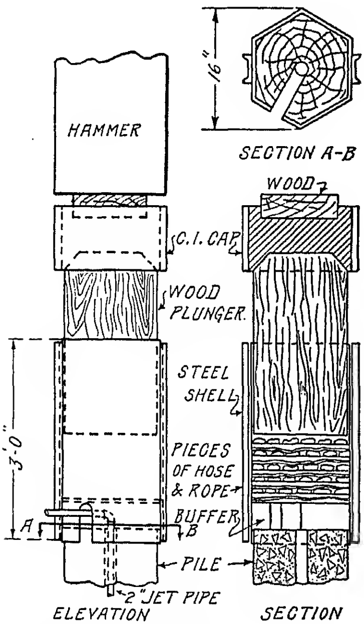


Fig. 58. Cushion Head for Driving Piles

Loading for Piles. The spacing for wood piles is generally 30 inches on centers. The loading of wood piles, with 12-inch butts, driven through wet, loose soil to a good bearing, is taken usually at 10 to 12 tons per pile. When driven through a firm soil the loading may be increased to 15 to 20 tons. Under the same conditions of soil, concrete piles 16 inches in diameter at the top and tapering to 8 or 10 inches at

with the wood pile is worth considering. In general, the requirements of the work to be done should be carefully noted before the type of pile is selected.

The cost of wood piles varies, depending on the size and length of the piles, and on the section of the country in which the piles are bought. Usually piles can be bought of lumber dealers at 10 to 20 cents per linear foot for all ordinary lengths; but very long piles will cost more. The cost of driving piles is variable, ranging from 2 or 3 cents to 12 or 15 cents per linear foot. A great many piles have been driven for which the contract price ranged from 20 cents to 30 cents per linear foot of pile driven. The length of the pile driven is the full length of the pile left in the work after cutting it off at the level desired for the cap.

The contract price for concrete piles about 16 inches in diameter and 25 to 30 feet long is approximately $1.00 per linear foot. When a price of $1.00 per linear foot is given for a pile of this size and length, the price will generally be somewhat reduced for a longer pile of the same diameter. Concrete piles have been driven for 70 cents per linear foot, and perhaps less; and again, they have cost much more than the approximate price of $1.00 per linear foot.

Piles for the Charles River Dam. The first piles driven for the Cambridge, Massachusetts, conduit of the Charles River dam were on the Cambridge shore. On January 1, 1907, 9,969 piles had been driven in the Boston and Cambridge cofferdams, amounting to 297,000 linear feet. Under the lock, the average length of the piles, after being cut off, was 29 feet; and under the sluices, 31 feet 4 inches. The specifications called for piles to be winter-cut from straight, live trees, not less than 10 inches in diameter at the butt when cut off in the work, and not less than 6 inches in diameter at the small end. The safe load assumed for the lock foundations was 12 tons per pile, and for the sluices 7 tons per pile.

The *Engineering News formula* was used in determining the bearing power of the piles. The piles under the lock walls were driven very close together; and, as a result, many of them rose during the driving of adjacent piles, and it was necessary to re-drive these piles.*

breakwater, piles 70 to 85 feet were used. These piles were in one length, single sticks. Toward the inner end of the breakwater, lengths of 100 to 110 feet were required. Single sticks of this length could not be secured, and it was therefore necessary to resort to splicing. After a trial of several methods, it was found that a splice made by means of a 10-inch wrought-iron pipe was most satisfactory. When the top of the first pile had been driven to within three feet of the water, it was trimmed down to 10 inches in diameter. On this end was placed a piece of 10-inch wrought-iron pipe 10 inches long. The lower end of the top pile was trimmed the same as the top of the first pile, and, when raised by the leads, was fitted into the pipe and driven until the required penetration was reached. The piles were cut off $4\frac{1}{2}$ feet below the surface of the water, by a circular saw mounted on a vertical shaft.*

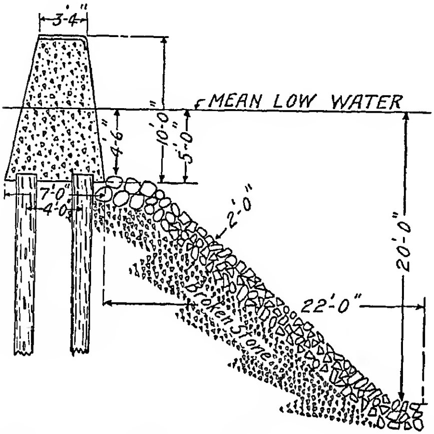


Fig. 59. Section of New Sea Wall, Annapolis, Maryland

COFFERDAMS, CRIBS, AND CAISSONS

Cofferdams. Foundations are frequently constructed through shallow bodies of water by means of cofferdams. These are essentially walls of clay confined between wood frames, the walls being sufficiently impervious to water so that all water and mud within the walled space may be pumped out and the soil excavated to the desired depth. It is seldom expected that a cofferdam can be constructed which will be so impervious to water that no pumping will be required to keep it clear; but when a cofferdam can be kept clear with a moderate amount of pumping, the advantages are so great

*Proceedings of the Engineers' Club of Philadelphia, Vol. XXIII, No. 3.

depth, say 5 to 10 feet, into which sheet piling may be driven. The sheet piles are driven as closely together as possible. The bottom of each pile, when made of wood, is beveled so as to form a wedge which tends to force it against the pile previously driven, Fig. 50. In this way a fairly tight joint between adjacent piles is obtained. Larger piles, *a*, Fig. 60, made of squared timber, are first driven to act as guide piles. These are connected by waling strips, *b*, Fig. 60, which are bolted to the guide piles and which serve as guides for the sheet piling, *c*, Fig. 60. The space between the two rows of sheet piling is filled with puddle, which ordinarily consists chiefly of clay. It is found that if the puddling material contains some gravel, there is less danger that a serious leak will form and enlarge. Numerous cross braces or tie-rods, *d*, Fig. 60, must be used to prevent the walls of sheet piling from spreading when the puddle is being packed between them. The width of the puddle wall is usually made to vary between three feet and ten feet, depending upon the depth of the water. When the sheet piling obtains a firm footing in the subsoil, it is comparatively easy to make the cofferdam water-tight; but when the soil is very porous so that the water soaks up from under the lower edge of the cofferdam, or when, on the other hand, the cofferdam is to be placed on a bare ledge of rock, or when the rock has only a thin layer of soil over it, it becomes exceedingly difficult to obtain a water-tight joint at the bottom of the dam. Excessive leakage is sometimes reduced by a layer of canvas or tarpaulin which is placed around the outside

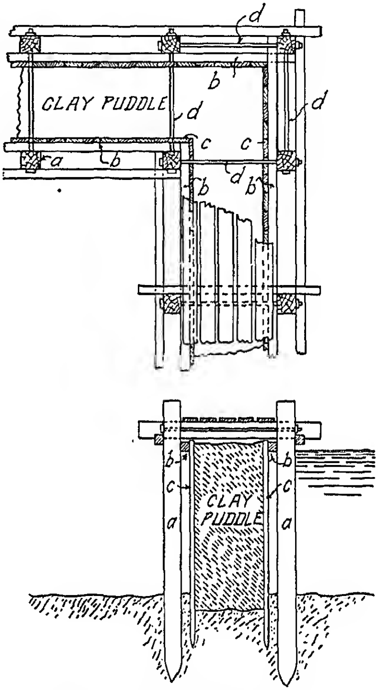


Fig. 60. Plan and Cross Section of a Cofferdam

of the base of the cofferdam, and which is held in place by stones laid on top of it. Brush, straw, and similar fibrous materials are used in connection with earth for stopping the cracks on the outside of the dam, and are usually effective, provided they are not washed away by a swift current.

Although cofferdams can readily be used at depths of 10 feet, and have been used in some cases at considerably greater depth, the difficulty of preventing leakage, on account of the great water pressure at the greater depths, usually renders some other method preferable when the depth is much, if any, greater than 10 feet.

Cribs. A crib is essentially a framework (called a *bird-cage* by the English) which is made of timber, and which is filled with stone to weight it down. Such a construction is used only when the entire timber work will be perpetually under water. The timber framework must, of course, be so designed that it will safely support the entire weight of the structure placed upon it. The use of such a crib necessarily implies that the subsoil on which the crib is to rest is sufficiently dense and firm to withstand the pressure of the crib and its load without perceptible yielding. It is also necessary for the subsoil to be leveled off so that the crib itself shall be not only level but also shall be so uniformly supported that it is not subjected to transverse stresses which might cripple it. This is sometimes done by dredging the site until the subsoil is level and sufficiently firm. Some of this dredging may be avoided through leveling up low spots by depositing loose stones which will imbed themselves in the soil and furnish a fairly firm subsoil. Although such methods may be tolerated when the maximum unit loading is not great—as for a breakwater or a wharf—it is seldom that a satisfactory foundation can be thus obtained for heavy bridge piers and similar structures.

Caissons. *Open Type.* A caisson is literally a box; and an open caisson is virtually a huge box which is built on shore and launched in very much the same way as a vessel, and which is sunk on the site of the proposed pier, Fig. 61. The box is made somewhat larger than the proposed pier, which is started on the bottom of the box. The sinking of the box is usually accomplished by the building of the pier inside of the box, the weight of the pier lowering it until it reaches the bed prepared for it on the subsoil. The preparation of

this bed involves the same difficulties and the same objections as those already referred to in the adoption of cribs. The bottom of the box is essentially a large platform made of heavy timbers and planking. The sides of the caissons have sometimes been made so that they are merely tied to the bottom by means of numerous tie-rods extending from the top down to the extended platform at the bottom, where they are hooked into large iron rings. When the pier is complete above the water line so that the caisson is no longer needed, the tie-rods may be loosened by unscrewing nuts at the top.

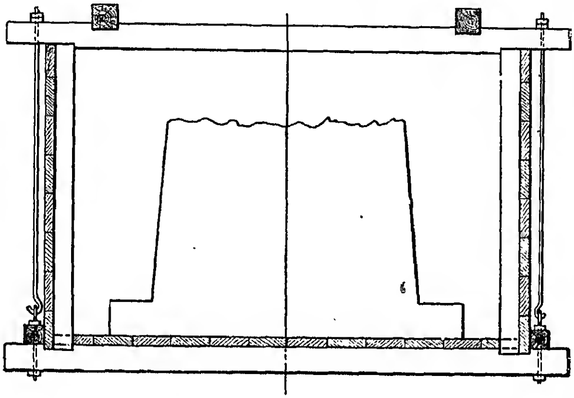


Fig. 61. Section of Open Caisson

The rods may then be unhooked, and nearly all the timber in the sides of the caisson will be loosened and may be recovered.

Hollow-Crib Type. The foundation for a pier is sometimes made in the form of a box with walls several feet in thickness, but with a large opening or well through the center. Such piers may be sunk in situations where there is a soft soil of considerable depth through which the pier must pass before it can reach the firm subsoil. In such a case, the crib or caisson, which is usually made of timber, may be built on shore and towed to the site of the proposed pier. The masonry work may be immediately started; and as the pier sinks into the mud, the masonry work is added so that it is always considerably above the water line, Fig. 62. The deeper the pier sinks, the greater will be the resistance of the subsoil, until, finally, the weight of the uncompleted pier is of itself insufficient to cause it to sink further. At this stage, or even earlier, dredging

bucket, through the interior well. The removal of the earth from the center of the subsoil on which the pier is resting will cause the mud and soft soil to flow toward the center, where it is within reach of the dredge. The pressure of the pier accomplishes this. The deeper the pier sinks, the greater is its weight and the greater its pressure on the subsoil, although this is somewhat counteracted by the constantly increasing friction of the soil around the outside of the pier. Finally, the pier will reach such a depth, and the subsoil will be so firm, that even the pressure of the pier is not sufficient to force any more loose soil toward the central well. The interior well may then be filled solidly with concrete, and thus the entire area of the base of the pier is resting on the subsoil, and the unit pressure is probably reduced to a safe figure for the subsoil at that depth.

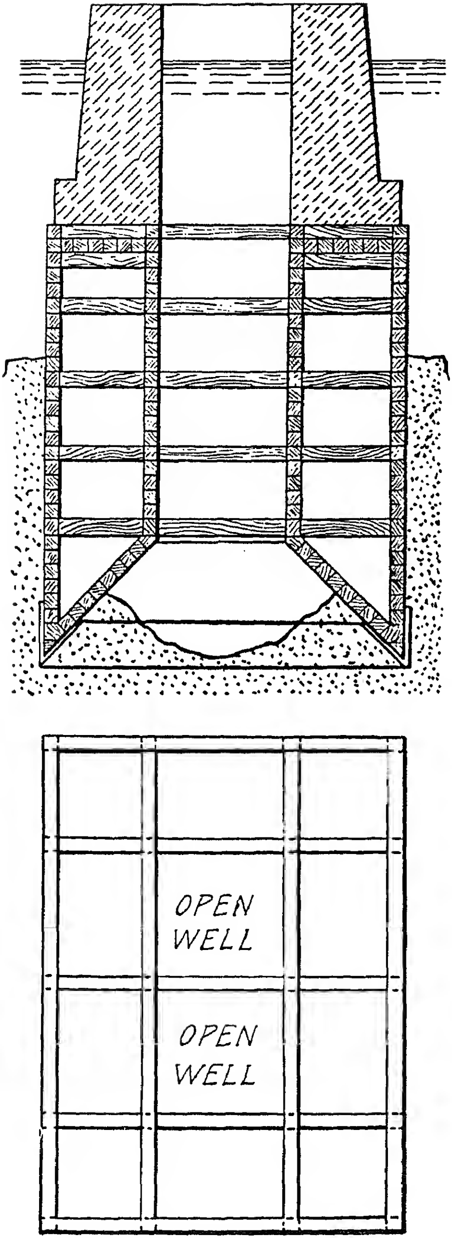


Fig. 62. Hollow Crib Material

This principle was adopted in the Hawkesbury bridge in Australia, which was sunk to a depth of 185 feet below high water—a depth which

would have been impracticable for the pneumatic caisson method described later. In this case, the caissons were made of iron, elliptical in shape, and about 48 feet by 20 feet. There were three tubes 8 feet in diameter through each caisson. At the bottom, these tubes flared out in bell-shaped extensions which formed sharp cutting edges with the outside line of the caisson. These bell-mouthed extensions thus forced the soil toward the center of the wells until the material was within reach of the dredging buckets.

This method of dredging through an opening is very readily applicable to the sinking of a comparatively small iron cylinder. As it sinks, new sections of the cylinder can be added; while the dredge, working through the cylinder, readily removes the earth until the subsoil becomes so firm that the dredge will not readily excavate it. Under such conditions the subsoil is firm enough for a foundation, and it is then only necessary to fill the cylinder with concrete to obtain a solid pier on a good and firm foundation.

One practical difficulty which applies to all of these methods of sinking cribs and caissons is the fact that the action of a heavy current in a river, or the meeting of some large obstruction, such as a boulder or large sunken log, may deflect the pier somewhat out of its intended position. When such a deflection takes place, it is difficult, if not impossible, to force the pier back to its intended position. It therefore becomes necessary to make the pier somewhat larger than the strict requirements of the superstructure would demand, in order that the superstructure may have its intended alignment, even though the pier is six inches or even a foot out of its intended position.

Pneumatic Type. A pneumatic caisson is essentially a large inverted box on which a pier is built, and inside of which work may be done because the water is forced out of the box by compressed air. If an inverted tumbler is forced down into a bowl of water, the large air space within the tumbler gives some idea of the possibilities of working within the caisson. If the tumbler is forced to the bottom of the bowl, the possibilities of working on a river bottom are somewhat exemplified. It is, of course, necessary to have a means of communication between this working chamber and the surface; and it is likewise necessary to have an *air lock* through

The process of sinking resembles, in many points, that described for the previous type. The caisson is built on shore, is launched, and is towed to its position. Sometimes, for the sake of economy, provided timber is cheap, that portion of the pier from the top of the working chamber to within a few feet below the low-water line may be built as a timber crib and filled with loose stone or gravel merely to weight it down. This method is usually cheaper than masonry; and the timber, being always under water, is durable. As in the previous instance, the caisson sinks as the material is removed

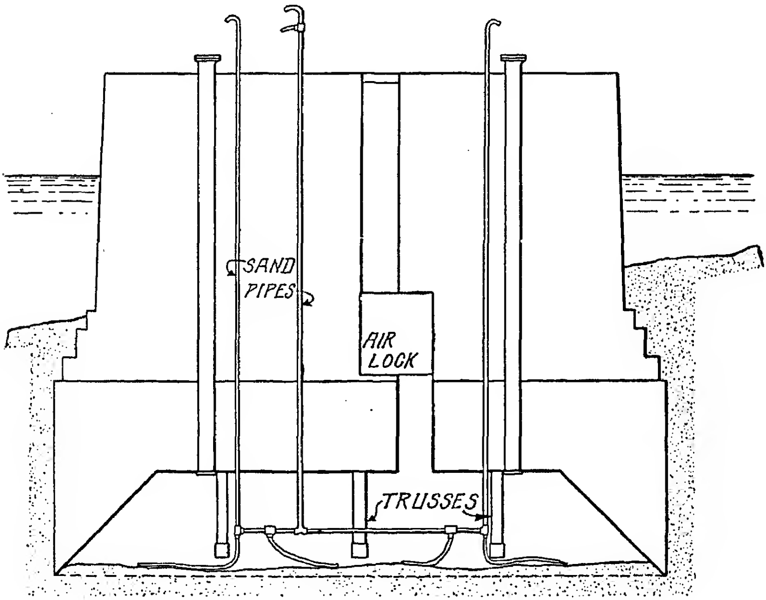


Fig. 63. Outline of Pneumatic Caisson

from the base, the sinking being assisted by the additional weight on the top. The only essential difference between the two processes consists in the method of removing the material from under the caisson. The greatest depth to which such a caisson has ever been

in such an air pressure, and even then four hours' work per day, in two shifts of two hours each, is considered a good day's work at these depths. The workmen are liable to a form of paralysis which is called *caisson disease*, and which, especially in those of weak constitution or intemperate habits, will result in partial or permanent disablement and even death.

In Fig. 63 is shown an outline, with but few details, of the pneumatic caisson used for a large bridge over the Missouri River near Blair, Nebraska. The caisson was constructed entirely of timber, which was framed in a fashion somewhat similar to that shown in greater detail in Fig. 62. The soil was very soft, consisting chiefly of sand and mud, which was raised to the surface by the operation of mud pumps that would force a stream of liquid mud and sand through the smaller pipes, which are shown passing through the pier. The larger pipes near each side of the pier were kept closed during the process of sinking the caisson and were opened only after the pier had been sunk to the bottom and the working chamber was being filled with concrete, as described below. These extra openings facilitated the filling of the working chamber with concrete. Near the center of the pier is an air lock with the shafts extending down to the working chamber and up to the surface. The structure of the caisson was considerably stiffened by the use of three trusses in order to resist any tendency of the caisson to collapse.

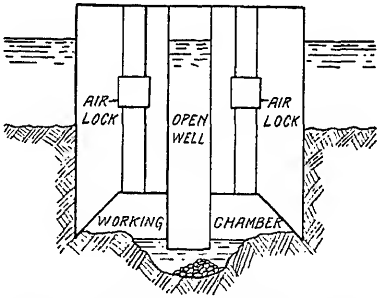


Fig. 64. Combination of Pneumatic Caisson and Open-Well Methods

A caisson is necessarily constructed in a very rigid manner, the timbers being generally 12- by 12-inch and laid crosswise in alternate layers, which are thoroughly interlocked. An irregularity in the settling may often be counteracted by increasing the rate of excavation under one side or the other of the caisson, so that the caisson will be guided in its descent in that direction.

A great economy in the operation of the compressed-air locks is afforded by combining the pneumatic process with the open-well process already described, by maintaining a pit in the center of the

caisson. A draft tube which is as low as the cutting edge of the caisson prevents a blow-out of air into the central well. The material dug by the workmen in the caisson is thrown loosely into the central well or sump, from which it is promptly raised by the dredging machinery, Fig. 64. By the adoption of this plan, the air lock needs to be used only for the entrance and exit of the workmen to and from the working chamber.

When the caisson has sunk to a satisfactory subsoil, and the bottom has been satisfactorily cleaned and leveled off, the working chamber is at once filled with concrete. As soon as sufficient concrete has been placed to seal the chamber effectively against the entrance of water, the air locks may be removed, and then the completion of the filling of the chamber and of the central shaft is merely open-air work.

RETAINING WALLS

A retaining wall is a wall built to sustain the lateral pressure of earth. The pressure that will be exerted on the wall will depend on the kind of material to be supported, the manner of placing it, and the amount of moisture that it contains. Earth and most other granular masses possess some frictional stability. Loose soil or a hydraulic pressure will exert a full pressure; but a compacted earth, such as clay, may exert only a small pressure due to the cohesion in the materials. This cohesion cannot be depended upon to relieve the pressure against a wall, for the cohesion may be destroyed by vibration due to moving loads or to saturation. In designing a wall the pressure due to a granular mass or a semifluid without cohesion must always be considered.

Causes of Failure of Walls. There are three ways in which a masonry wall may fail: (1) by sliding along a horizontal plane; (2) by overturning or rotating; (3) by crushing of the masonry or its footing. These are the three points that must be considered in order to design a wall that will be successful in resisting an embankment. A wall, therefore, must be of sufficient size and weight to prevent the occurrence of sliding, rotation, or crushing.

Stability of Wall Against Sliding. Stability against sliding is

joint. A movement will occur when E equals fW, where f is the coefficient of friction. Let n be a number greater than unity, the factor of safety, then in order that there be no movement n must be sufficiently large so that nE equals fW. A common value for n is 2, but sometimes it is taken as low as $1\frac{1}{2}$. Substituting 2 for n,

$$2E = fW$$

$$W = \frac{2E}{f} \qquad (6)$$

Average values of the coefficients of friction of masonry on masonry is 0.65; for masonry on dry clay, 0.50; for masonry on wet clay, 0.33; masonry on gravel, 0.60; masonry on wood, 0.50.

Stability Against Rotation. The stability against rotation of a wall is secured by making the wall of such dimension and weight that the resultant R of the external forces will pass through the base and well within the base, as shown in Fig. 65. Generally, in designing, the resultant is made to come within or at the edge of the middle third. The nearer the center of the base the resultant comes, the more evenly the pressure will be distributed over the foundation for the wall. When R passes through A, Fig. 65, the wall will fail by rotation. Methods for finding R will be demonstrated in another paragraph.

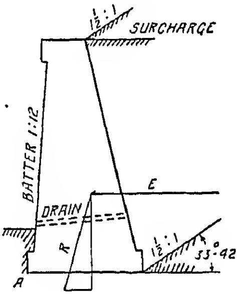


Fig. 65. Section of Retaining Wall

Stability Against Crushing. The compressive unit stresses in walls must not be greater than the unit stresses permitted for safe working loads of masonry (see pages 13, 54, Part I), when the wall is built on a stone foundation; but when it is built on clay, sand, or gravel the allowable pressure for such foundations must not be exceeded.

Foundations. The foundations for a retaining wall must be below the frost line, which is about three feet below the surface in a temperate climate, and deeper in a cold climate. The foundation should be of such a character that it will safely support the wall. If necessary, the soil should be tested to determine if it will safely support the wall.

The foundation should always be well drained. Many failures

a wall greatly increases the stresses in the wall. When water freezes behind a wall it usually causes it to bulge out, which is the first step in the failure of the wall. On a clay foundation the friction is greatly reduced by the clay becoming thoroughly soaked with water. On page 151 it is shown that the difference of the coefficients of friction of masonry on dry clay and wet clay is 0.17. There are different ways of draining a fill behind a retaining wall. The method shown in Fig. 38 for drainage often can be used. Pipes two to four inches in diameter are often built in the wall, as shown in Fig. 65.

DESIGN OF WALL

In designing a retaining wall the dimensions of the section of a wall are generally assumed and then the section investigated graphically to see if it is right for the conditions assumed. There are theoretical formulas for designing walls which will be given. In designing a wall, the student is advised to first make the section according to the formulas and investigate it graphically.

Fill Behind Wall. The fills behind the walls are sometimes made horizontal with the top of the wall; at other times the fill is sloped back from the top of the wall, as shown in Fig. 65. When there is a slope to be supported, the wall is said to be surcharged, and the load to be supported is greater than for a horizontal fill.

Faces of Wall. The front or face of retaining walls is usually built with a batter. This batter often varies from less than an inch per foot in height to more than an inch per foot. The rear face may be built either straight, with a batter, or stepped up. A wall should never be less than $2\frac{1}{2}$ feet to 3 feet in width on top, unless the wall is a very small one. In that case, probably a width of 12 to 18 inches would be sufficient for the top.

Width of Base. The following values for the width of the base of a wall are taken from Trautwine's Handbook, and are based on the fill behind the wall being placed loosely, as is usually the case.

Wall of cut stone or of first-class large-ranged rubble, in mortar
.35 of its entire vertical height
Wall of good common mortar rubble or brick
.4 of its entire vertical height
Wall of well-laid dry rubble

retaining walls. A wall built of a 1:3:6 concrete should be equal in strength to a wall built of cut stone or large-ranged rubble. In heavy walls large stones, twenty-five to fifty per cent in volume, are often placed in the concrete. This, usually, greatly reduces the cost of the wall and does not weaken the wall if the stones are properly placed.

Value of Study of Existing Walls. When designing a retaining wall, all existing walls in that vicinity should be examined to determine their dimensions and to discover if they have been successfully designed. Often, existing walls will give more information to an engineer than he will obtain by a theoretical or graphical study.

Pressure Behind Wall. The development of the formulas for finding the pressure behind a wall is a long, complicated theory, and the demonstration will not be given here. The formulas given are those usually found in textbooks. They are based on the Rankine theory, which considers that the earth is a granular mass with an assumed angle of repose of 1.5 to 1, which in degrees is 33° 42′. In applying this method it is immaterial whether the forces representing the earth pressure are considered as acting directly upon the back of the wall, or are considered as acting on a vertical plane passing through the extreme back of the footing. In the latter case, the force representing the lateral earth pressure must be combined with (1) the vertical force representing the weight of the earth prism between the back of the wall and the vertical plane considered; and (2) combined with the vertical force representing the weight of the wall itself.

In the formulas for determining pressures behind a wall let E equal total pressure against rear face of wall on a unit length of wall; W equal weight of a unit volume of the earth; h equal height of wall; and ϕ equal angle of repose.

When the upper surface of the earth is horizontal, the equation is

$$E = \tan^2\left(45° \; \frac{\phi}{2}\right)\frac{Wh^2}{2} \qquad (7)$$

Since the angle of repose for the earth behind the wall has been taken as 33° 42′, Equation (7) may be reduced to the following form by substituting the value of the tangent of the angle in the equation

$$E = .286\,\frac{Wh^2}{2} \qquad \textbf{(7a)}$$

When a wall must sustain a surcharge at the slope of 1.5 to 1, the equation is

$$E = \tfrac{1}{2} \cos \phi \, Wh^2 \qquad \textbf{(7b)}$$

or

$$E = .833 \frac{Wh^2}{2} \qquad \textbf{(7c)}$$

The force E is applied at one-third the height of the wall, measured from the bottom, but for surcharged wall it is applied at one-third of the height of a plane that passes just behind the wall. This is clearly shown in the different figures illustrating retaining walls.

The direction of the center of pressure E is assumed as being parallel to the top of the earth back of the wall. The angle of the surcharge is generally made 1.5 to 1.

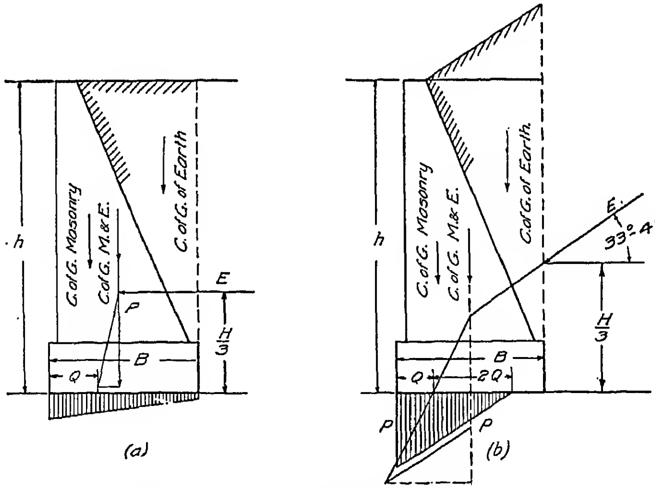


Fig. 66. Diagrams Showing Pressures on Foundations

Example. What is the pressure per foot of length of a wall 18 feet high, earth weighing 100 pounds per cubic foot, if the fill is level with the top of the wall.

Solution. Substituting in equation (7a),

$$E = .286 \frac{Wh^2}{2}$$

$$= .286 \frac{100 \times 18^2}{2}$$

NOTE: When P equals the vertical component of the resultant pressure on the base, B is the full width of the base in feet, and Q is the distance from the toe to where the force P cuts the base.

When Q is equal to or greater than $\frac{B}{3}$

$$\text{Pressure at the toe} = (4B - 6Q)\frac{P}{B^2} \qquad \text{(7d)}$$

$$\text{Pressure at the heel} = (6Q - 2B)\frac{P}{B^2} \qquad \text{(7e)}$$

When Q is less than $\frac{B}{3}$

$$\text{Pressure at the toe} = \frac{2P}{3Q} \qquad \text{(7f)}$$

Example. Design a retaining wall to support an embankment 20 feet high, the top of the fill being level with the top of the wall; the face of the wall to be vertical, the back to slope.

Solution. Draw an outline of the proposed section, Fig. 67, and then investigate the section to see if it has sufficient strength to support the embankment. Make the base .45 of the height of the wall.

$$\text{Width of base} = 20 \text{ feet} \times .45$$
$$= 9.0 \text{ feet}$$

Assume the width at the top at 3 feet, and find the pressure E at the back, substituting in equation (7a), and apply that pressure at $\frac{H}{3}$.

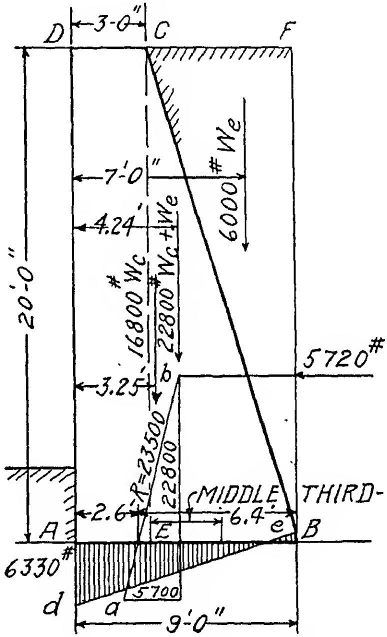


Fig. 67. Design Diagram for Retaining Wall

$$E = .286\frac{Wh^2}{2}$$
$$= .286\frac{100 \times 20^2}{2}$$
$$= 5{,}720$$

P is found by dividing the wall into a rectangle and a triangle and finding the weights and the center of gravity of each, and also that of the triangle of

of the wall, and then finding the combined weights and the center of the wall and earth. Assume that the weight of the masonry is s per foot and the earth 100 pounds per cubic foot, and consider of wall as being one foot in length. The center of gravity of the be obtained thus:

Section	Area	Moment (Arm)	Moment (Area)
$A\ E\ C\ D$	60.0	1.5	90.0
$E\ B\ C$	60.0	5.0	300.0
	120.0		390.0

om A to center of gravity $= 390 \div 120 = 3.25$ feet
wall per lineal foot $= 120 \times 140 = 16{,}800$ pounds
noment about $A = 16{,}800 \times 3.25 = 54{,}600$ foot-pounds

ravity of the earth is at one-third of the distance from the back of the e, or 7.0 feet from the face of the wall.

earth per lineal foot, $\dfrac{6 \times 20}{2} \times 100 = 6{,}000$ pounds

oment about $A = 6{,}000 \times 7 = 42{,}000$ foot-pounds

n of the resultant is determined by dividing the sum of the static its by the sum of the weights:

$$\frac{54{,}600 + 42{,}000}{16{,}800 + 6{,}000} = \frac{96{,}600}{22{,}800} = 4.24 \text{ feet}$$

ce the line E to meet the vertical line passing through the combined avity. On this vertical line lay off the value of P, which is 22,800, venient scale. At the lower end of P draw a line parallel to line E line lay off the value of E, which is 5,720. Draw line $a\,b$, which tant of the two forces. This line cuts the base at a scaled distance from the toe, which is a point without the middle third of the base, is less than $\dfrac{B}{3}$.

g in equation (7f) for the condition when Q is less than $\dfrac{B}{3}$ we have

$$\text{Pressure at toe} = \frac{2 \times 22{,}800}{3 \times 2.6} = 6{,}330 \text{ pounds}$$

f $A\,d$, at any convenient scale, equal to 6,330 pounds and on the base tance equal to $3\,Q = 7.8$ feet. Through this point draw $d\,e$ and scale own from e to the base line, which is 1,000 pounds.

xamination of this section of wall shows that the pressure of the toe sive for an ordinary foundation, such as clay. At the heel there is 1,000 pounds. This uplift would be overcome by the friction of

20 feet high, although the resultant does not come in the middle third of the base. The weight of the material in this problem was taken as 140 pounds per cubic foot. This is the weight of 1:3:6 concrete. If closely laid stone were used, the weight of the masonry per cubic foot would be increased to 160 pounds at least. This increased weight would bring the resultant within the middle third.

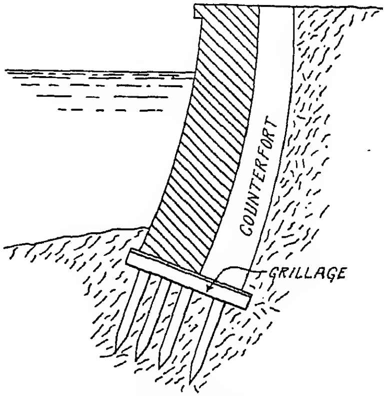


Fig. 68. Retaining Wall with Curved Cross Section

The wall will next be investigated for stability against sliding on its base. Suppose that the wall is to be built on dry clay. The horizontal thrust E is 5,720 pounds, the total weight is 22,800 pounds, and the coefficient of friction of masonry on dry clay is .50.

Substituting in equation (6),

$$22{,}800 = \frac{2 \times 5{,}720}{.50}$$

$$11{,}400 = 11{,}440$$

The approximate equality of the equation shows that there is a factor of two against sliding on such a base. On a base of wet clay the factor against sliding would be less than one and a quarter and it would be necessary to secure the wall against sliding in some way.

Types of Walls. In Fig. 68 is shown a type of wall that has sometimes been used. The tendency to slide outward at the bottom, and even the tendency to overturn, is resisted by making the lower course with the joints inclined towards the rear. This method of construction makes the joints nearer perpendicular to the line of pressure than in a vertical wall. The weakness of this type of wall is that water running down the face of it will enter the joints and produce an additional pressure to that of the earth. There is also the danger of this water freezing behind the wall and causing the wall to bulge out.

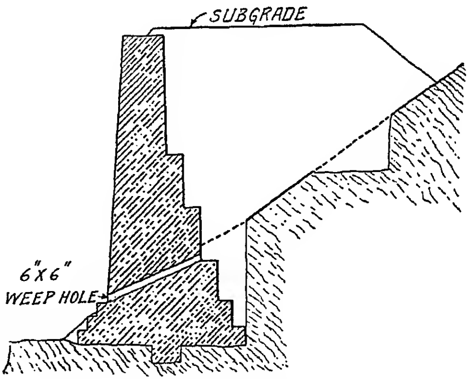


Fig. 69. Retaining Wall for Railroad Embankment

It should be noted that the width of the base is nearly one-half the height but that this width is only carried up a short distance. The back is stepped, therefore it receives the assistance of the maximum vertical pressure of the earth on the horizontal steps. The wall is anchored to the foundation by a projection below the base of the wall.

BRIDGE PIERS AND ABUTMENTS

PIERS

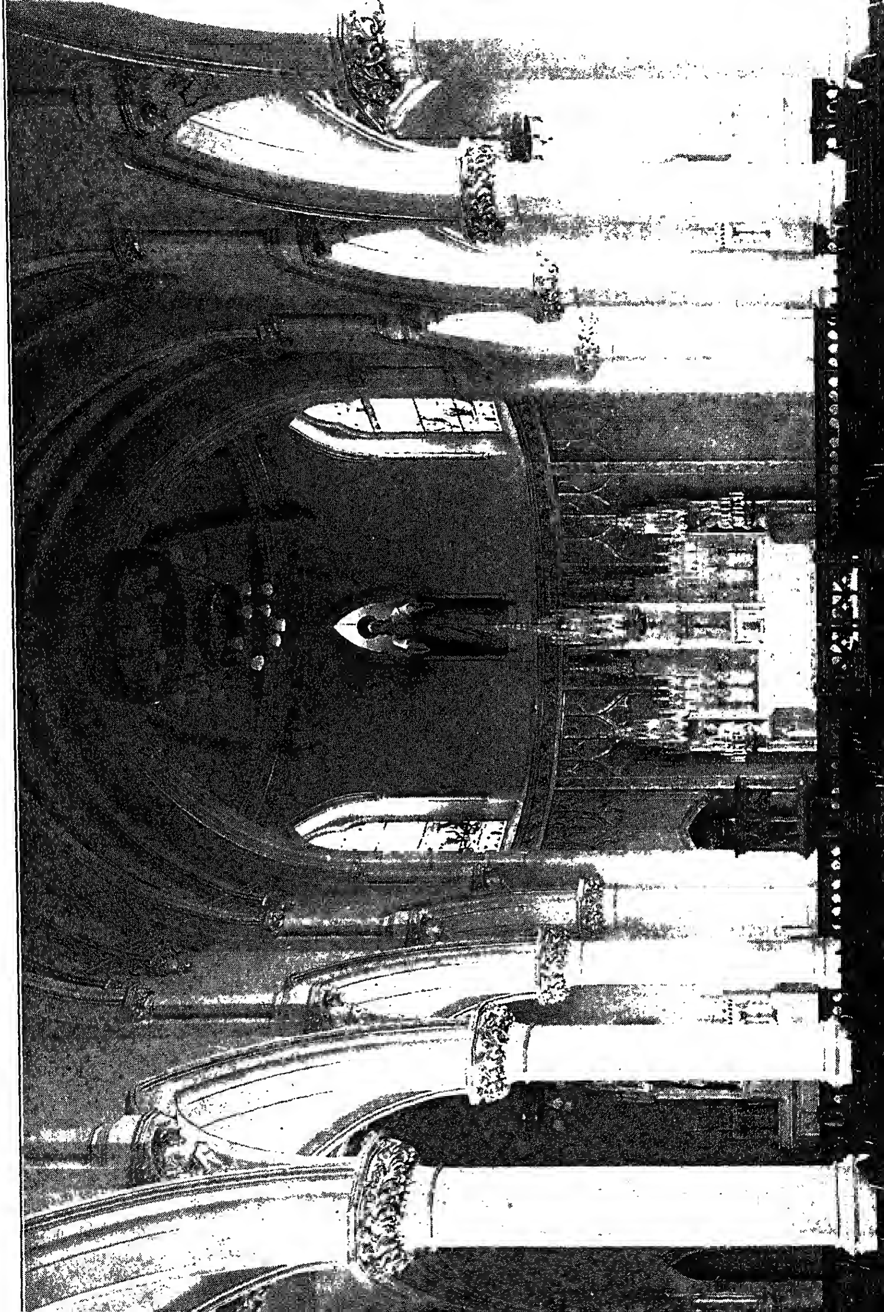
Location. The outline design of a long bridge which requires several spans involves many considerations:

(1) If the river is navigable, at least one deep and wide channel must be left for navigation. The placing of piers, the clear height of the spans above high water, and the general plans of all bridges over navigable rivers are subject to the approval of the United States Government.

(2) A long bridge always requires a solution of the general question of few piers and long spans, or more piers and shorter spans. No general solution of the question is possible, since it depends on the required clear height of the spans above the water, on the required depth below the water for a suitable foundation, and on several other conditions (such as swift current, etc.) which would influence the relative cost of additional piers or longer spans. Each case must be decided according to its particular circumstances.

(3) Even the general location of the line of the bridge is often determined by a careful comparison, not only of several plans for a given crossing, but even a comparison of the plans for several locations.

Sizes and Shapes. The requirements for the bridge seats for the ends of the two spans resting on a pier are usually such that a pier with a top as large as thus required, and with a proper batter to the faces, will have all the strength necessary for the external forces acting on the pier. For example, the channel pier of one of the large railroad bridges crossing the Mississippi River was capped by a course of stonework 14 feet wide and 29 feet long, besides two semicircles with a radius of 7 feet. The footing of this pier was 30 feet wide by 70 feet long, and the total height from subsoil to



top was about 170 feet. This pier, of course, was unusually large. For trusses of shorter span, the bridge seats are correspondingly smaller. The elements which affect stability are so easily computed that it is always proper, as a matter of precaution, to test every pier designed to fulfil the other usual requirements, to see whether it is certainly safe against certain possible methods of failure. This is especially true when the piers are unusually high.

The requirements for supporting the truss are, fortunately, just such as give the pier the most favorable formation so that it offers the least obstruction to the flow of the current in the river. In other words, since the normal condition is for a bridge to cross a river at right angles, the bridge piers are always comparatively long, in the direction of the river, and narrow in a direction perpendicular to the flow of the current. The rectangular shape, however, is modified by making both the upper and the lower ends pointed. The pointing of the upper end serves the double purpose of deflecting the current, and thus offers less resistance to the flow of the water; and it also deflects the floating ice and timber, so that there is less danger of the formation of a jam during a freshet. The lower end should also be pointed in order to reduce the resistance to the flow of the water The ends of the piers are sometimes made semicircular, but a better plan is to make them in the form of two arcs of circles which intersect at a point.

Causes of Failure. The forces tending to cause a bridge pier to fail in a direction perpendicular to the line of the bridge include the action of wind on the pier itself, on the trusses, and on a train which may be crossing the bridge. They will also include the maximum possible effect of floating ice in the river and of the current due to a freshet. It is not at all improbable that all of these causes may combine to act together simultaneously. The least favorable condition for resisting such an effect is that produced by the weight of the bridge, together with that of a train of empty cars, and the weight of the masonry of the pier above any joint whose stability is in question. The effects of wind, ice, and current will tend to make the masonry slide on the horizontal joints. They will also increase the pressure on the subsoil on the downstream end of the foundation of a pier. They will tend to crush the masonry on the

Another possible cause of failure of a bridge pier arises from forces parallel with the length of the bridge. The stress produced on a bridge by the sudden stoppage of a train thereon, combined with a wind pressure parallel with the length of the bridge, will tend to cause the pier to fail in that direction, Fig. 70. Although these forces are never so great as the other external forces, yet the resisting power of the pier in this direction is so very much less than that in the other direction, that the factor of safety against failure is probably less, even if there is no actual danger under any reasonable values for these external forces.

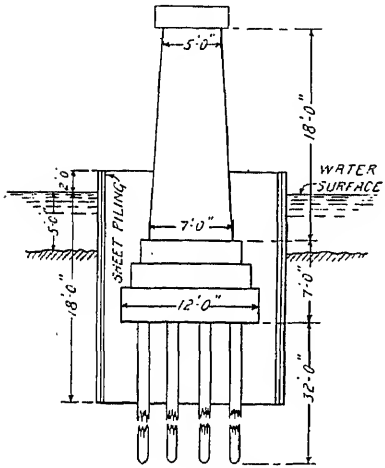


Fig. 70. Bridge Pier

Abutment Piers. A pier is usually built comparatively thin in the direction of the line of the bridge, because the forces tending to produce overturning in that direction are usually very small. When a series of stone arches are placed on piers, the thrusts of the two arches on each side of a pier nearly balance each other, and it is only necessary for the pier to be sufficiently rigid to withstand the effect of an eccentric loading on the arches; but if, by any accident or failure, one arch is destroyed, the thrust on such a pier is unbalanced and the pier will probably be overturned by the unbalanced thrust of the adjoining arch. The failure of that arch would similarly cause the failure of the succeeding pier and arch. On this account a very long series of arches usually includes an abutment pier for every fourth or fifth pier. An abutment pier is one which has sufficient thickness to withstand the thrust of an arch, even though it is not balanced by the thrust of an arch on the other side of the pier. Abutment piers are chiefly for arch bridges; but all piers should have sufficient rigidity in the direction of the line of the bridge so that any possible thrust which may come from the action of a truss of the bridge may be resisted, even if there is no counterbalancing thrust from an adjoining truss.

ABUTMENTS

Requirements of Design. The term abutment usually implies not only a support for the bridge, but also what is virtually a retaining wall for the bank behind it. In the case of an arch bridge, the thrust of the arch is invariably so great that there is never any chance that the pressure of the earth behind the abutment will throw the abutment over, and therefore the abutment never needs to be designed as a retaining wall in this case; but when the abutment supports a truss bridge which does not transmit any horizontal thrust through the bridge, the abutment must be designed as a retaining wall. The conditions of stability for such structures have already been discussed. This principle of the retaining wall is especially applicable if the abutment consists of a perfectly straight wall. There are other forms of abutments which tend to prevent failure as a retaining wall, on account of their design.

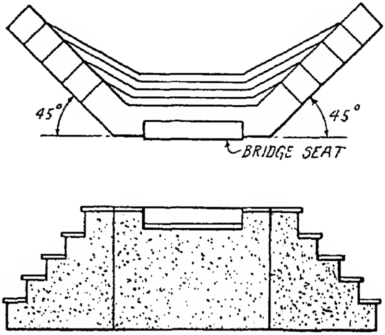


Fig. 71. Typical Abutment with Flaring Wing Walls

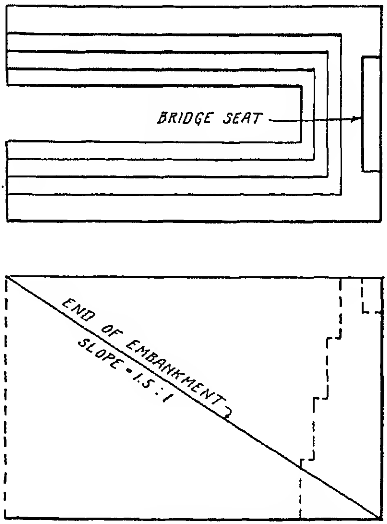


Fig. 72. U-Shaped Abutment

Abutments with Flaring Wing Walls. These are constructed substantially as shown in Fig. 71. The wing walls make an angle of about 30° to 45° with the face of the abutment, and the height decreases at such a rate that it will just catch the embankment formed behind it, the slopes of the embankment probably being at the rate of 1.5 : 1. If the bonding of the wing walls, and especially the bonding at the junction of the

wing walls will act virtually as counterforts and will materially assist in resisting the overturning tendency of the earth. The assistance given by these wing walls will be much greater as the angle between the wing walls and the face becomes larger.

U-Shaped Abutments. These consist of a head wall and two walls which run back perpendicular to the head wall, Fig. 72. This form of wall is occasionally used, but the occasions are rare when such a shape is necessary or desirable.

T-Shaped Abutments. As the name implies, these consist of a head wall which has a core wall extending perpendicularly back from the center. The core wall serves to tie the head wall and prevent its overturning. Of course such an effect can be produced only by the adoption of great care in the construction of the wall, so that the bonding is very perfect and so that the wall has very considerable tensile strength; otherwise the core wall could not resist the overturning tendency of the earth pressure against the rear face of the abutment.

CULVERTS

The term culvert is usually applied to a small waterway which passes under an embankment of a railroad or a highway. The term is confined to waterways which are so small that standard plans are prepared which depend only on the assumed area of waterway that is required. Although the term is sometimes applied to arches having a span of 10 or 15 feet, or even more, the fact that the structures are built according to standard plans justifies the use of the term culvert as distinguished from a structure crossing some perennial stream where a special design for the location is made. The term culvert, therefore, includes the drainage openings which may be needed to drain the hollow on one side of an embankment, even though the culvert is normally dry.

Types of Culverts. Culverts are variously made of cast iron, wrought iron, and tile pipe, wood, stone blocks with large cover plates of stone slabs, stone arches, and plain and reinforced concrete; still another variety is made by building two side walls of stone and making a cover plate of old rails.

Culverts made of wood should be considered as temporary, on account of the inevitable decay of the wood in the course of a few

years. When wood is used, the area of the opening should be made much larger than that actually required, so that a more permanent culvert of sufficient size may be constructed inside of the wood culvert before it has decayed. For present purposes, the discussion of the subject of Culverts will be limited to those built of stone and concrete.

Stone Box Culverts. The choice of stone as a material for culverts should depend on the possibility of obtaining a good quality of building stone in the immediate neighborhood. Frequently temporary trestles are used when good stone is unobtainable, with the idea that after the railroad is completed, it will be possible to transport a suitable quality of building stone from a distance and build the culvert under the trestle. The engineer should avoid the mistake of using a poor quality of building stone for the construction of even a culvert, simply because such a stone is readily obtainable. Since a culvert always implies a stream of water which will have a scouring action during floods, it is essential that the side walls of culverts should have an ample foundation, which is sunk to such a depth that there is no danger that it will be undermined. There are cases where a bed of quicksand has been encountered, and where the cost of excavating to a firmer soil would be very large. In such a case, it is generally possible to obtain a sufficient foundation by constructing a platform or grillage of timber, which underlies the entire culvert, beneath the floor of the culvert. Of course, timber should not be used for the foundation, except in cases where it will always be underneath the level of the ground water and will therefore always be wet. If the soil has a character such that it will be easily scoured, the floor of the culvert between the side walls should be paved with large pebbles, so as to protect it from scouring action. At both ends of the culvert, there should always be built a vertical wall, which should run from the floor of the culvert down to a depth that will certainly be below any possible scouring influence, in order that the side walls and the flooring of the culvert cannot possibly be undermined.

The above specifications apply to all forms of stone culverts, and even to arch culverts, and in the cases of the larger arch culverts the precautions in these respects should be correspondingly

which are from 2 to 4 feet apart, they are sometimes capped with large flagstones covering the span between the walls. The thickness of the cover stone is sometimes determined by an assumption as to the transverse strength of the stone, and by applying the ordinary theory of flexure. The application of this theory depends on the assumption that the neutral axis for a rectangular section is at the center of depth of the stone, and that the modulus of elasticity for tension and compression is the same. Although these assumptions are practically true for steel and even wood, they are far from being true for stone. It is therefore improper to apply the theory of flexure to stone slabs, except on the basis of moduli of rupture which have been experimentally determined from specimens having substantially the same thickness as the thickness proposed. Also, on account of the variability of the actual strength of stones, though nominally of the same quality, a very large factor of safety over the supposed ultimate strength of the stone should be used.

The maximum moment at the center of a slab one foot wide equals $\frac{1}{8} Wl$, in which W equals the total load on the width of one foot of the slab, and l equals the span of the slab, in feet; but by the principles of mechanics, this moment equals $\frac{1}{6} Rh^2$, in which R equals the modulus of transverse strength, in pounds per square foot; and h equals the thickness of the stone, in feet. Placing these two expressions equal to each other, and solving for h, we find:

$$h^2 = \frac{6}{8} \times \frac{Wl}{R}$$

$$h = \sqrt{\frac{3}{4} \times \frac{Wl}{R}} \tag{8}$$

Example. Assume that a culvert is covered with 6 feet of earth weighing 100 pounds per cubic foot. Assume a live load on top of the embankment equivalent to 500 pounds per square foot, in addition; or that the total load on top of the slab is equivalent to 1,100 pounds per square foot of slab. Assume that the slab is to have a span l of 4 feet. Then the total load W on a section of the slab one foot wide will be 1,100 × 4 or 4,400 pounds. Assume that the stone is sandstone with an average ultimate modulus of 525 pounds per per square inch (see Table XVI), and that the safe value R is 55 pounds per square inch, or 144 × 55 pounds per square foot. Substituting these values in the above equation for h, we find that h equals 1.29 feet, or 15.5 inches.

The above problem has been worked out on the basis of the live

be correspondingly decreased. It should be noted that in the above formula the thickness of the stone h varies as the square root of the span; therefore, for a span of 3 feet (other things being the same as above), the thickness of the stone h equals $15.5\times\sqrt{\frac{3}{4}}$ or 13.4 inches. For a span of 2 feet, the thickness should be $15.5\times\sqrt{\frac{2}{4}}$ or 11.0 inches.

Owing to the uncertainty of the true transverse strength of building stone, as has already been discussed in the design of offsets for footings (see pages 117, 118), no precise calculation is possible; and therefore many box culverts are made according to empirical

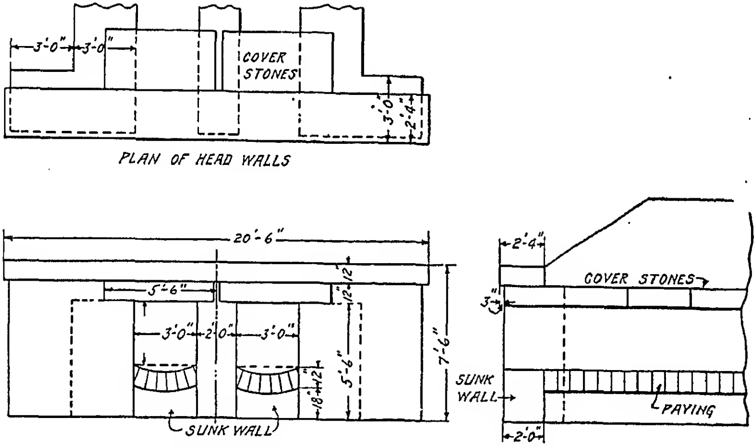


Fig. 73. Detail Diagrams for a Double Box Culvert

rules, which dictate that the thickness shall be 10 inches for a 2-foot span, 13 inches for a 3-foot span, and 15 inches for a 4-foot span. These values are slightly less than those computed above.

Although a good quality of granite, and especially of bluestone flagging, will stand higher transverse stresses than those given above for sandstone, the rough rules just quoted are more often used, and are, of course, safer. When it is desired to test the safety of stone already cut into slabs of a given thickness, their strength may be computed from Equation (8), using the values for transverse stresses as already given in Table XVI.

Double Box Culverts. A box culvert with a stone top is gen-

course, be possible to obtain thicker stones which would safely carry the load over a considerably greater span. Therefore, when the required culvert area demands a greater width of opening than 4 feet, and when this type of culvert is to be used, the culvert may be made as illustrated in Fig. 73, by constructing an intermediate wall which supports the ends of the two sets of cover stones forming the top. A section and elevation of a double box culvert of 3 feet span and a net height of 3 feet is shown in Fig. 73. The details of the wing walls and end walls are also shown. The double box culvert illustrated in Fig. 74 has two spans, each of 4 feet. The stone used

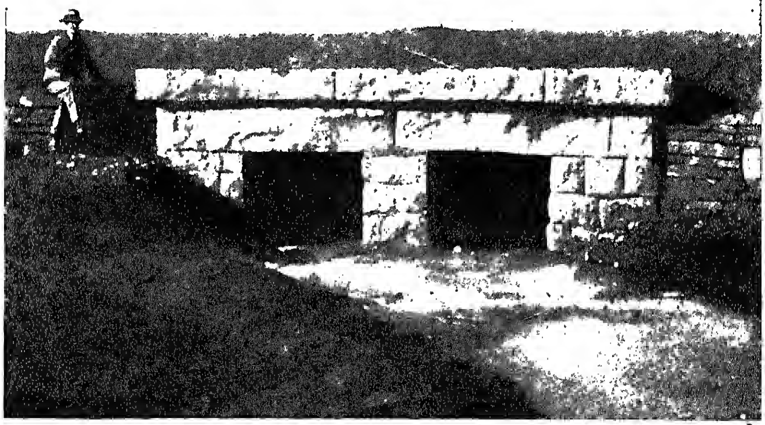


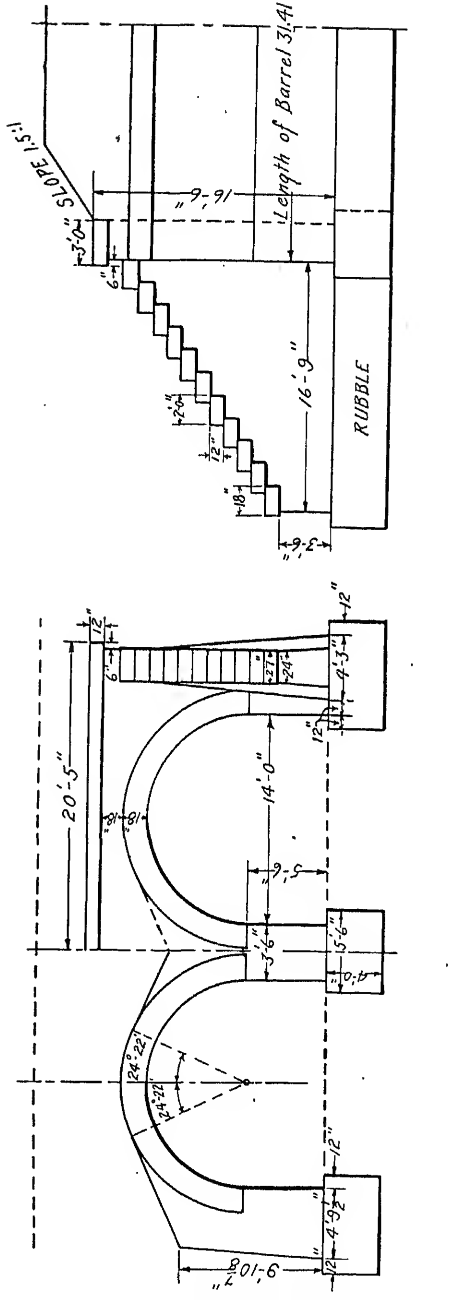
Fig. 74. Double Box Culvert. Openings 4 by 3 Feet

was a good quality of limestone. The cover stones were made 15 inches thick.

Box culverts are sometimes constructed as dry masonry—that is, without the use of mortar. This should never be done, except for very small culverts and when the stones are so large and regular that they form close, solid walls with comparatively small joints. A dry wall made up of irregular stones cannot withstand the thrusts which are usually exerted by the subsequent expansion of the earth embankment above it.

Plain Concrete Culverts. Culverts may be made of plain con-

much the same general dimensions as those already given for stone box culverts. They have a great advantage over stone culverts in that they are essentially monoliths. If the side walls and top are formed in one single operation, the joint between the side walls and top becomes a source of additional strength, and the culverts are therefore much better than similar culverts made of stone. The formula developed above, Equation (8), for the thickness of the concrete slab on top of a box culvert may be used, together with the modulus of transverse strength as given for concrete in Table XVI. This formula will apply, even though the slab for the cover of the culvert is laid after the side walls are built, and the slab is considered as merely resting on the side walls. If the side walls and top are constructed in one operation so that the whole structure is actually a monolith, it may be considered that there is that much additional strength in the structure; but it would hardly be wise to reduce the thickness of the concrete slab by depending upon the continuity be-



Arch Culverts. Stone arches are frequently used for culverts in cases where the span is not great, and in which the design of the culvert, except for some small details regarding the wing walls, depends only on the span of the culvert. The design of some arch culverts used on the Atchison, Topeka & Santa Fé Railway, Figs. 73 and 75, is copied from a paper presented to the American Society of Civil Engineers by A. G. Allan, Assoc. M. Am. Soc. C. E. The span of these arches is 14 feet, and the thickness at the crown is 18 inches. A photograph of one of these arch culverts, which shows also many other details, is reproduced in Fig. 76.

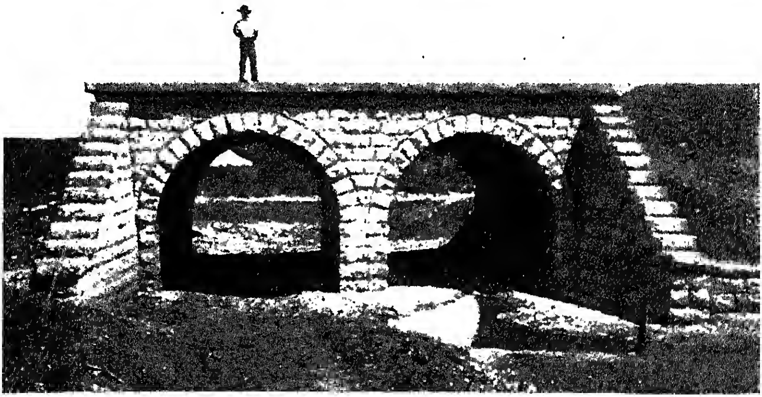


Fig. 76. Double Arch Culvert. Openings, 14 by 5½ Feet

End Walls. The ends of a culvert are usually expanded into end walls for the retention of the embankment. For the larger culverts, this may develop into two wing walls which act as retaining walls to prevent the embankment from falling over into the bed of the stream. An end wall is especially necessary on the upstream end of the culvert, so as to avoid the danger that the stream will scour the bank and work its way behind the culvert walls. The end wall is also carried up above the height of the top of the culvert, in order to guard still further against the washing of earth from the embankment over the end of the culvert into the stream below. All of these

should be tamped thoroughly, and the excavation filled with cinders, broken stone, gravel, or brickbat, to within four inches (or whatever thickness of slab is to be used) of the top of the grade. The foundation should be thoroughly rammed, and by using gravel or cinders to make this foundation, a very firm surface can be secured.

Side drains should be put in at convenient intervals where outlets can be secured. The foundation is sometimes omitted, even in cold climates, if the soil is porous. Walks laid on the natural soils have proven, in many cases, to be very satisfactory.

At the Convention of the National Cement Users' Association, held at Buffalo, New York, in 1908, the Committee on Sidewalks, Streets, and Floors presented the following specifications for sidewalk foundations:

The ground base shall be made as solid and permanent as possible. Where excavations or fills are made, all wood or other materials which will decompose shall be removed, and replaced with earth or other filling like the rest of the foundation. Fills of clay or other material which will settle after heavy rains or deep frost should be tamped, and laid in layers not more than six inches in thickness, so as to insure a solid embankment which will remain firm after the walk is laid. Embankments should not be less than $2\frac{1}{2}$ feet wider than the walk which is to be laid. When porous materials, such as coal ashes, granulated slag, or gravel, are used, underdrains of tile should be laid to the curb drains or gutters, so as to prevent water accumulating and freezing under the walk and breaking the block.

Concrete Base. The concrete for the base of walks is usually composed of 1 part Portland cement, 3 parts sand, and 5 parts stone or gravel. Sometimes, however, a richer mixture is used, consisting of 1 part cement, 2 parts sand, and 4 parts broken stone; but this mixture seems to be richer than what is generally required. The concrete should be thoroughly mixed and rammed, Fig. 77, and cut into uniform blocks. The size of the broken stone or gravel should not be larger than one inch, varying in size down to $\frac{1}{4}$ inch, and free from fine screenings or soft stone. All stone or gravel under $\frac{1}{8}$ inch is considered sand.

Fig. 77. Square Tamper

The thickness of the concrete base will depend upon the location, the amount of travel, or the danger of being broken by frost. The usual thickness in residence districts is 3 inches, with a wearing thickness of 1 inch, making a total of 4 inches, Fig. 78. In business sections, the walks vary from 4 to 6 inches in total thickness, in which the finishing coat should not be less than $1\frac{1}{4}$ inches thick. The concrete base is cut into uniform blocks.

The lines and grades given for walks by the Engineer should be carefully followed. The mold strips should be firmly blocked and kept perfectly straight to the height of the grade given. The walks usually are laid with a slope of $\frac{1}{4}$ inch to the foot toward the curb.

The blocks are usually from 4 to 6 feet square, but sometimes they are made much larger than these dimensions. The joints made

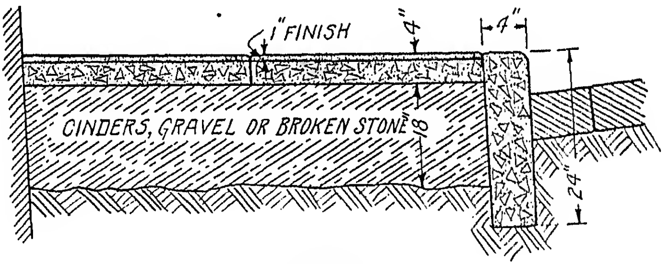


Fig. 78. Concrete Sidewalk and Curb

by cutting the concrete should be filled with dry sand, and the exact location of these joints should be marked on the forms. The cleaver or spud that is used in making the joints should not be less than $\frac{1}{8}$ of an inch or over $\frac{1}{4}$ of an inch in thickness.

Top Surface. The wearing surface usually consists of 1 part Portland cement and 2 parts crushed stone or good, coarse sand—all of which will pass through a $\frac{1}{4}$-inch mesh screen—thoroughly mixed so that a uniform color will be secured. This mixture is then spread over the concrete base to a thickness of one inch, this being done before the concrete of the base has set or become covered with dust. The mortar is leveled off with a straightedge, and smoothed down

been in place from two to five hours and is partially set. This final floating is done first with a wood float, and afterwards with a metal float or trowel. The top surface is then cut directly over the cuts made in the base, care being taken to cut entirely through the top and base all around each block. The joint is then finished with a jointer, Fig. 79, and all edges rounded or beveled. Care should be taken, in the final floating or finishing, not to overdo it, as too much working will draw the cement to the surface, leaving a thin layer of neat cement, which is likely to peel off. Just before the floating, a very thin layer of *dryer*, consisting of dry cement and sand mixed in the proportion of one to one, or even richer, is frequently spread over the surface;

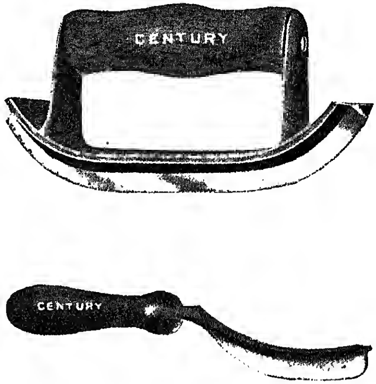


Fig. 79. Jointers

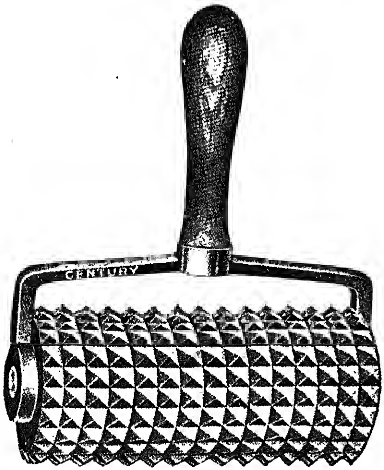


Fig. 80. Brass Dot Roller

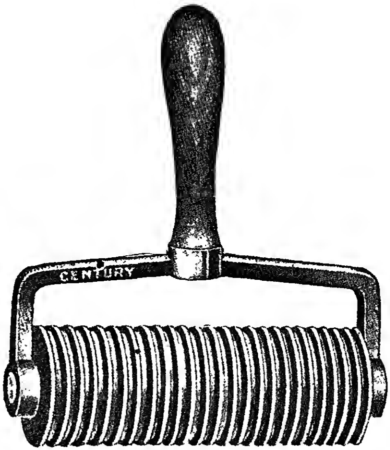


Fig. 81. Brass Line Roller

but this is generally undesirable, as it tends to make a glossy walk. A dot roller or line roller, Figs. 80 and 81, may be employed to

already referred to, the Committee on Sidewalks, Floors, and Streets recommended the following specifications for the top coat:

> Three parts high-grade Portland cement and five parts clean, sharp sand, mixed dry and screened through a No. 4 sieve. In the top coat, the amount of water used should be just enough so that the surface of the walk can be tamped, struck off, floated, and finished within 20 minutes after it is spread on the bottom coat; and when finished, it should be solid and not quaky.

In the January, 1907, number of *Cement*, Mr. Albert Moyer, Assoc. M. Am. Soc. C. E., in discussing the subject of cement sidewalk pavements, gives specifications for monolithic slab for paving purposes. For an example of this construction, he gives the pavement around the Astor Hotel, New York:

> As an alternative, and instead of using a top coat, make one slab of selected aggregates for base and wearing surface, filling in between the frames concrete flush with established grade. Concrete to be of selected aggregates, all of which will pass through a $\frac{3}{4}$-inch mesh sieve; hard, tough stones or pebbles, graded in size; proportions to be 1 part cement, $2\frac{1}{2}$ parts crushed hard stone screenings or coarse sand, all passing a $\frac{3}{4}$-inch mesh, and all collected on a $\frac{1}{4}$-inch mesh. Tamped to an even surface, prove surface with straightedge, smooth down with float or trowel, and in addition a natural finish can be obtained by scrubbing with a wire brush and water while concrete is "green", but after final set.

Seasoning. The wearing surface must be protected from the rays of the sun by a covering which is raised a few inches above the pavement so as not to come in contact with the surface. After the pavement has set hard, sprinkle freely two or three times a day for a week or more.

Cost. The cost of concrete sidewalks is variable. The construction at each location usually requires only a few days' work; and the time and expense of transporting the men, tools, and materials make an important item. One of the skilled workmen should be in charge of the men, so that the expense of a foreman will not be necessary. The amount of walk laid per day is limited by the amount of surface that can be floated and troweled in a day. If the surfacers do not work overtime, it will be necessary to stop concreting in the middle of the afternoon, so that the last concrete placed will be in condition to finish during the regular working hours. The work of concreting may be continued considerably later in the afternoon if a drier concrete is used in mixing the top coat, and only enough water is used so that the surface can be floated and finished

soon after being placed. The men who have been mixing, placing, and ramming concrete can complete their day's work by preparing and ramming the foundations for the next day's work.

The contract price for a well-constructed sidewalk 4 to 5 inches in thickness, with a granolithic finish, will vary from 15 cents to 30 cents per square foot.

CONCRETE CURB

The curb is usually built just in advance of the sidewalk. The foundation is prepared similarly to that of walks. The curb is divided into lengths similar to that of the walk; and the joints between the blocks, and also between the walk and the curb, are made similar to the joints between the blocks of the walk. The concrete is generally composed of 1 part Portland cement, 3 parts sand, and 5 parts stone, although a richer mixture is sometimes used. A facing of mortar or granolithic finish on the part exposed to wear will improve the wearing qualities of the curb.

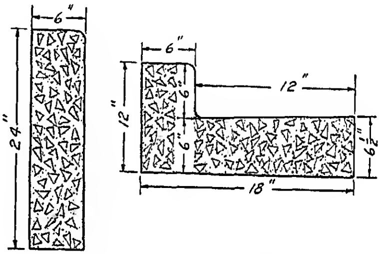


Fig. 82. Typical Curb Sections

Types of Curbing. There are two general types of curb used—a curb rectangular in section, and a combined curb and gutter; both types are shown in Fig. 82. The foundation for either type is constructed in the same manner. Both these types of curb are made in place or molded and set in place like stone curb, but the former method is preferable. A metal corner is sometimes laid in the exposed edge of the curb to protect it from wear.

Construction. The construction of the rectangular section is a simple process, but requires care to secure a good job. This is usually about 7 inches wide and from 20 to 30 inches deep. After the foundation has been properly prepared, the forms are set in place. Fig. 83 shows the section of a curb 7 inches wide and 24 inches deep, and the forms as they are often used. The forms for the front and back each consist of three planks $1\frac{7}{8}$ inches thick and 8 inches wide, and are surfaced on the side next the concrete. They are held in

stakes are kept from spreading by a clamp. A sheet-iron plate $\frac{1}{4}$ inch thick is inserted every 6 feet, or at whatever distance the joints are made. After the concrete has been placed and rammed, and has set hard enough to support itself, the plate and front forms are removed, and the surface and top are finished smooth with a trowel, and with other tools such as shown in Figs. 84, 85, and 86. The joint is usually plastered over, and acts as an expansion joint. The forms on the back are not removed until the concrete is well set. If a mortar or granolithic finish is used, a piece of sheet iron is placed in the form one inch from the facing, and mortar is placed between the sheet iron and the front form, and the coarser concrete is placed back of the sheet iron, Fig. 87. The sheet iron is then withdrawn and the two concretes thoroughly tamped.

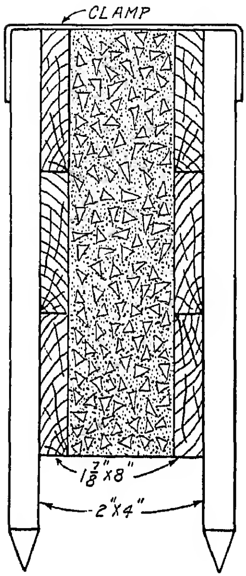


Fig. 83. Forms for Constructing Curb

Fig. 84. Curb Edger

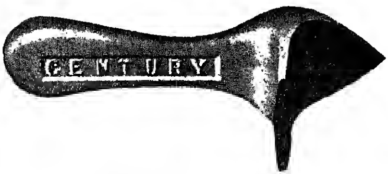


Fig. 85. Radius Tool

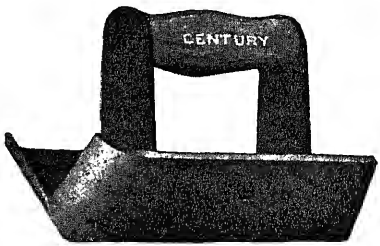


Fig. 86. Inside-Angle Tool

Fig. 87 shows the section of a combined curb and gutter, and the forms that are necessary for its construction. This combination is often laid on a porous soil without any special foundation, with fair

PITTSBURGH SERVICE BUILDING FOR FORD MOTOR COMPANY
Courtesy of Condron and Company, Chicago

results. A $1\frac{7}{8}$-inch plank 12 inches wide is used for the back fo
and is held in place at the bottom by pegs. The front form con
of a plank $1\frac{7}{8}$ by 6 inches, and is held in place by pegs. Before
concrete is placed, two sheet-iron plates, cut as shown in the fig
are placed in the forms, six feet to eight feet apart. After the
crete for the gutter and the lower part of the curb is placed
rammed, a $1\frac{7}{8}$-inch plank is placed against these plates and hel
place by screw clamps, Fig. 87. The upper part of the curb is
molded. When the concrete is set enough to stay in place, the f
forms and plates are removed, and the surface is treated in the s
manner as described for the other type of curb.

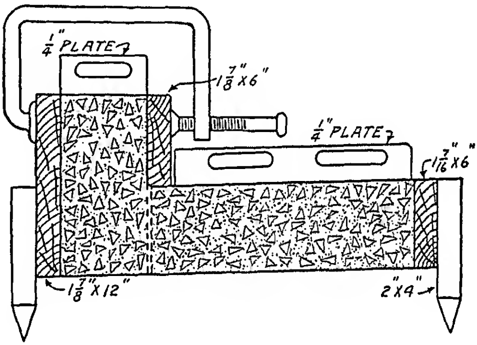
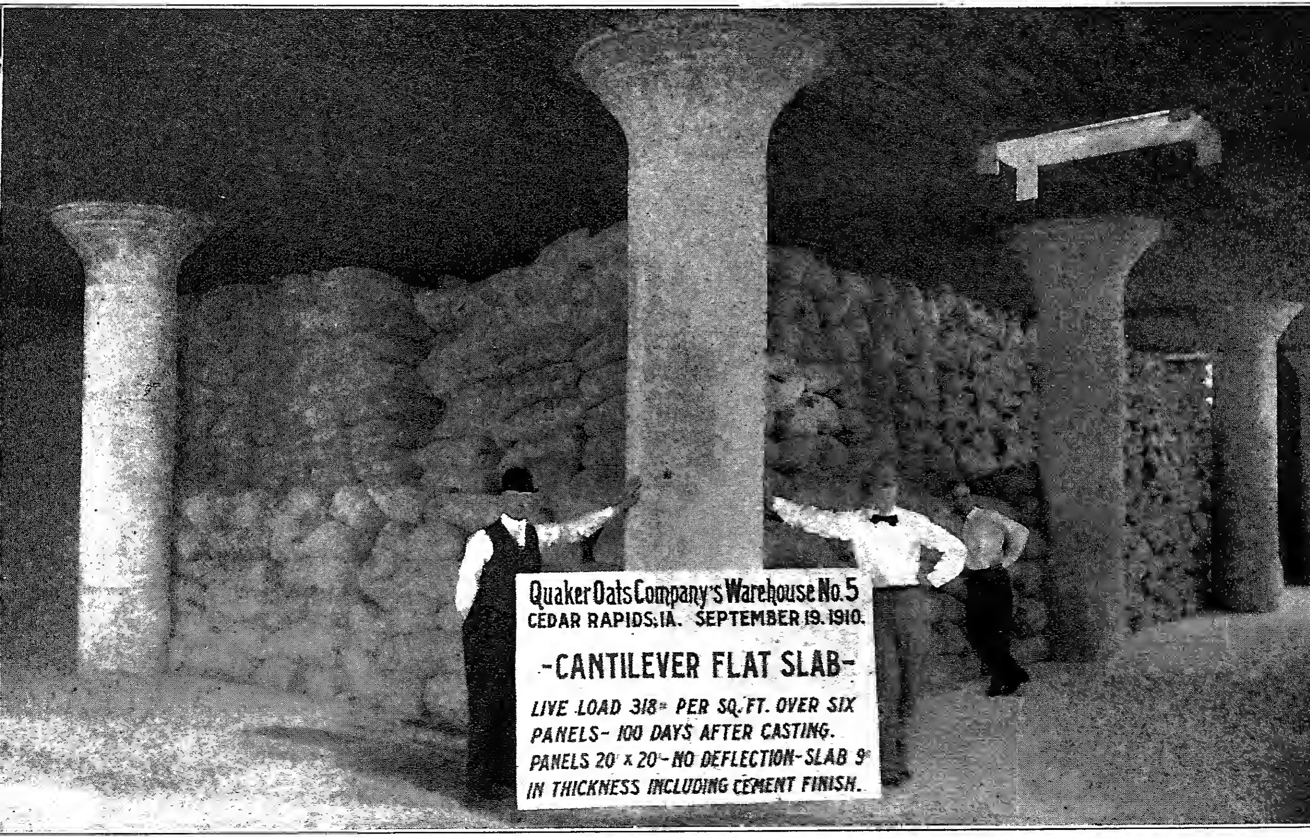


Fig. 87. Forms for Curb and Gutter

Cost. The cost of concrete curb will depend upon the co
tions under which it is made. Under ordinary circumstances
contract price for rectangular curbing 6 inches wide and 24 in
deep will be about 60 cents per linear foot; or 80 cents per linear
for curb 8 inches wide and 24 inches deep. Under favorable c
tions on large jobs, 6-inch curbing can be constructed for 40 cen
45 cents per linear foot. These prices include the excavation th
required below the street grade.

The cost of the combined curb and gutter is about 10 to 2
cent more than that of the rectangular curbing. In addition to
ing a larger surface to finish, the combined curb and gutter req
more material, and therefore more work, to construct it.



TEST OF QUAKER OATS COMPANY'S CONCRETE BUILDING

MASONRY AND REINFORCED CONCRETE

PART III

REINFORCED CONCRETE BEAM DESIGN

GENERAL THEORY OF FLEXURE

Introduction. The theory of flexure in reinforced concrete is exceptionally complicated. A multitude of simple rules, formulas, and tables for designing reinforced-concrete work have been proposed, some of which are sufficiently accurate and applicable under certain conditions. But the effect of these various conditions should be thoroughly understood. Reinforced concrete should not be designed by "rule-of-thumb" engineers. It is hardly too strong a statement to say that a man is criminally careless and negligent when he attempts to design a structure, on which the safety and lives of people will depend, without thoroughly understanding the theory on which any formula he may use is based. The applicability of all formulas is so dependent on the quality of both the steel and the concrete, as well as on many of the details of the design, that a blind application of a formula is very unsafe. Although the greatest pains will be taken to make the following demonstration as clear and plain as possible, it will be necessary to employ symbols, and to work out several algebraic formulas on which the rules for designing will be based. The full significance of many of the following terms may not be fully understood until several subsequent paragraphs have been studied:

SYMBOLS DEFINED

b = Breadth of concrete beam
d = Depth from compression face to center of gravity of the steel
A = Area of the steel
p = Ratio of area of steel to area of concrete above the center of gravity of the steel, generally referred to as *percentage of reinforcement*,
A

E_s = Modulus of elasticity of steel
E_c = *Initial* modulus of elasticity of concrete
$n = \frac{E_s}{E_c}$ = Ratio of the moduli
s = Tensile stress per unit of area in steel
c = Compressive stress per unit of area in concrete at the outer fiber of the beam
ϵ_s = Deformation per unit of length in the steel
ϵ_c = Deformation per unit of length in outer fiber of concrete
k = Ratio of dimension from neutral axis to center of compressive stresses to the total effective depth d
j = Ratio of dimension from steel to center of compressive stresses to the total effective depth d
x = Distance from compressive face to center of compressing stresses
ΣX = Summation of horizontal compressive stresses
M = Resisting moment of a section

Statics of Plain Homogeneous Beams. As a preliminary to the theory of the use of reinforced concrete in beams, a very brief discussion will be given of the statics of an ordinary homogeneous beam, made of a material whose moduli of elasticity in tension and compression are equal. Let AB, Fig. 88, represent a beam carrying a uniformly distributed load W; then the beam is subjected to transverse stresses.

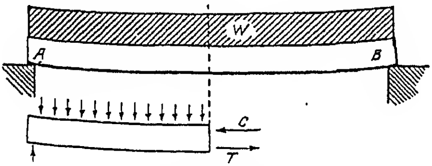


Fig. 88. Diagram of Beam Carrying Uniformly Distributed Load

Let us imagine that one-half of the beam is a "free body" in space and is acted on by exactly the same external forces; let us also assume forces C and T (acting on the exposed section), which are just such forces as are required to keep that half of the beam in equilibrium. These forces and their direction are represented in the lower diagram by arrows. The load W is represented by the series of small, equal, and equally spaced vertical arrows pointing downward. The reaction of the abutment against the beam is an upward force, shown at the left. The forces acting on a section at the center are the equivalent of the two equal forces C and T.

The force C, acting at the top of the section, must act toward the left, and there is therefore compression in that part of the section. Similarly, the force T is a force acting toward the right, and the fibers of the lower part of the beam are in tension. For our

case the resultant of the forces acting on a very large number of "fibers". The stress in the outer fibers is, of course, greatest. At the center of the height, there is neither tension nor compression. This is called the *neutral axis*, Fig. 89.

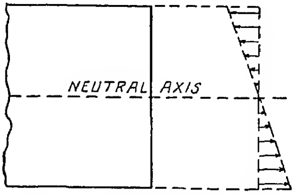


Fig. 89. Diagram Showing Position of Neutral Axis in Beam

Let us consider for simplicity a very narrow portion of the beam, having the full length and depth but so narrow that it includes only one set of fibers, one above the other, as shown in Fig. 90. In the case of a plain rectangular homogeneous beam, the elasticity being assumed equal for tension and compression, the stresses in the fibers would be as given in Fig. 89; the neutral axis would be at the center of the height, and the stress at the bottom and the top would be equal but opposite. If the section were at the center of the beam, with a uniformly distributed load, as indicated in Fig. 88, the shear would be zero.

A beam may be constructed of plain concrete; but its strength will be very small, since the tensile strength of concrete is comparatively insignificant. Reinforced concrete utilizes the great tensile strength of steel in combination with the compressive strength of concrete. It should be realized that two of the most essential qualities are *compression* and *tension*, and that, other things being equal, the cheapest method of obtaining the necessary compression and tension is the most economical.

Economy of Concrete for Compression. The ultimate compressive strength of concrete is generally 2,000 pounds, or over, per square inch. With a factor of safety of 4, a working stress of 500 pounds per square inch may be considered allowable. We may estimate that the concrete costs 20 cents per cubic foot, or $5.40 per cubic yard. On the other hand, we may estimate that the steel, placed in the work, costs about 3 cents per pound. It will weigh 480 pounds per cubic foot; therefore, the steel costs $14.40 per cubic foot, or 72 times as much as an equal volume of concrete or an equal

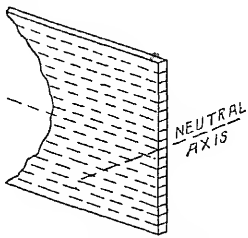


Fig. 90. Position of Neutral Axis in Narrow Beam

compressive stress of 16,000 pounds per square inch, which is 32 times the safe working load on concrete. Since, however, a given volume of steel costs 72 times an equal volume of concrete, the cost of a given compressive resistance in steel is $\frac{72}{32}$, or 2.25, times the cost of that resistance in concrete. Of course, the above assumed unit prices of concrete and steel will vary with circumstances. The advantage of concrete over steel for compression may be somewhat greater or less than the ratio given above, but the advantage is almost invariably with the concrete. There are many other advantages which will be discussed later.

Economy of Steel for Tension. The ultimate tensile strength of ordinary concrete is rarely more than 200 pounds per square inch. With a factor of safety of 4, this would allow a working stress of only 50 pounds per square inch. This is generally too small for practical use and certainly too small for economical use. On the other hand, steel may be used with a working stress of 16,000 pounds per square inch, which is 320 times that allowable for concrete. Using the same unit values for the cost of steel and concrete as given in the previous paragraph, even if steel costs 72 times as much as an equal volume of concrete, its real tensile value economically is $\frac{320}{72}$, or 4.44, times as great. Any reasonable variation from the above unit values cannot alter the essential truths of the economy of steel for tension and of concrete for compression. In a reinforced-concrete beam, the steel is placed in the tension side of the beam. Usually it is placed 1 to 2 inches from the outer face, with the double purpose of protecting the steel from corrosion or fire, and also to better insure the union of the concrete and the steel. But the concrete below the steel is not considered in the numerical calculations. The concrete between the steel and the neutral axis performs the very necessary function of transmitting the tension in the steel to the concrete. This stress is called *shear* and is discussed on page 207. Although the concrete in the lower part of the beam is, theoretically, subject to the tension of transverse stress and does actually con-

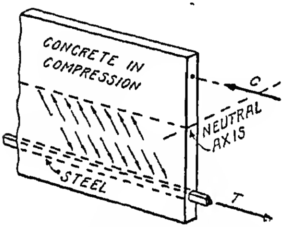


Fig. 91. Diagram Showing Transmission of Tension in Steel to Concrete

small, the proportion of the necessary tension which the concrete can furnish when the beam is heavily loaded is so very small that it is usually ignored, especially since such a policy is on the side of safety, and also since it greatly simplifies the theoretical calculations and yet makes very little difference in the final result. We may, therefore, consider that in a unit section of the beam, Fig. 91, the concrete above the neutral axis is subject to compression, and that the tension is furnished entirely by the steel.

Elasticity of Concrete in Compression. In computing the transverse stresses in a wood beam or steel I-beam, it is assumed that the modulus of elasticity is uniform for all stresses within the elastic limit. Experimental tests have shown this to be so nearly true that it is accepted as a mechanical law. This means that if a force of 1,000 pounds is required to stretch a bar .001 of an inch, it will require 2,000 pounds to stretch it, .002 of an inch. Similar tests have been made with concrete, to determine the law of its elasticity. Unfortunately, concrete is not so uniform in its behavior as steel. The results of tests are somewhat erratic. Many engineers have argued that the elasticity is so nearly uniform that it may be considered to be such within the limits of practical use. But all experimenters, who have tested concrete by measuring the proportional compression produced by various pressures, agree that the additional shortening produced by an additional pressure is greater at higher pressures than at low pressures.

A test of this sort may be made substantially as follows: A square or circular column of concrete at least one foot long is placed in a testing machine. A very delicate micrometer mechanism is fastened to the concrete by pointed screws of hardened steel. These points are originally at a known distance apart—say 8 inches. When the concrete is compressed, the distance between these points will be slightly less. A very delicate mechanism will permit this distance to be measured as closely as the ten-thousandth part of an inch, or to about $\frac{1}{100{,}000}$ of the length. Suppose that the various pressures per square inch, and the proportionate compressions, are as given in the following tabular form, which gives figures which are fairly representative of the behavior of ordinary

Pressure per Square Inch	Proportionate Compression
200 pounds	.00010 of total length
400 pounds	.00020 of total length
600 pounds	.00032 of total length
800 pounds	.00045 of total length
1,000 pounds	.00058 of total length
1,200 pounds	.00062 of total length
1,400 pounds	.00090 of total length
1,600 pounds	.00112 of total length

We may plot these pressures and compressions, Fig. 92, using any convenient scale for each. For example, for a pressure of 800 pounds per square inch, select the vertical line which is at the horizontal distance from the origin 0 of 800, according to the scale adopted. Scaling off on this vertical line the ordinate .00045, according to the scale adopted for compressions, we have the position of one point of the curve.. The other points are obtained similarly. Although the points thus obtained from the testing of a single block of concrete would not be considered sufficient to establish the law of the elasticity of concrete in compression, a study of the curves, which may be drawn through the series of points obtained for each of a large number of blocks, shows that these curves will average very closely to parabolas that are tangent to the initial modulus of elasticity, which is here represented in the diagram by a straight line running diagonally across the figure.

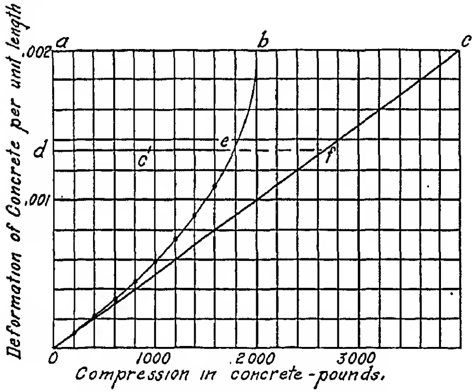


Fig. 92. Curve of Pressure and Compressions in Concrete

It was formerly quite common to base the computation of formulas on the assumption that the curve of compression is a parabola. The development of the theory is correspondingly complex, but it may be noted from Fig. 92 that for a compression of 600 or even 800 pounds per square inch, the parabolic curve is not very different from a straight line. A comparison of the results

often not greater than the uncertainty as to the true strength of the concrete. The straight-line theory will, therefore, be used exclusively in the demonstrations which follow.

Theoretical Assumptions. The theory of reinforced-concrete beams is based on the usual assumptions that:

(1) The loads are applied at right angles to the axis of the beam. The usual vertical gravity loads supported by a horizontal beam fulfill this condition.

(2) There is no resistance to free horizontal motion. This condition is seldom, if ever, exactly fulfilled in practice. The more rigidly the beam is held at the ends, the greater will be its strength above that computed by the simple theory. Under ordinary conditions the added strength is quite indeterminate; and is not allowed for, except in the appreciation that it adds indefinitely to the safety.

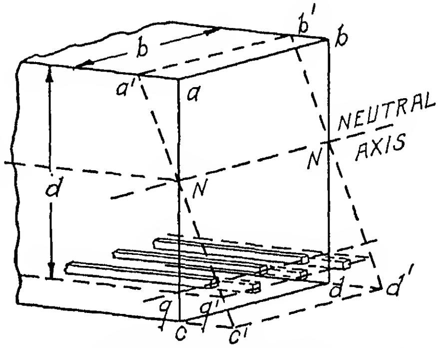


Fig. 93. Exaggerated Diagram Showing Plane Section of Beam Before and After Bending

(3) The concrete and steel stretch together without breaking the bond between them. This is absolutely essential.

(4) Any section of the beam which is plane before bending is plane after bending.

In Fig. 93 is shown, in a very exaggerated form, the essential meaning of assumption (4). The section *abcd* in the unstrained condition, is changed to the plane $a'b'd'c'$ when the load is applied. The compression at the top equals aa' equals bb'. The neutral axis is unchanged. The concrete at the bottom is stretched an amount equal to cc' equals dd', while the stretch in the steel equals gg'. The compression in the concrete between the neutral axis and the top is proportional to the distance from the neutral axis.

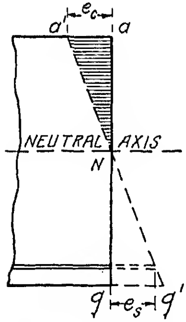


Fig. 94. Diagram Showing Side View of Beam with Reference to Deformation of Fibers

In Fig. 94 is given a side view of the beam, with special reference to the deformation of the fibers. Since the fibers between the neutral axis and the compressive face are compressed proportionally, then, if aa' represents the linear compression of the outer fiber, the

shaded lines represent, at the same scale, the compression of the intermediate fibers.

Summation of Compressive Forces. The summation of compressive forces evidently equals the sum of all the compressions, varying from zero to the maximum compressive stress c at the extreme upper fiber, where the linear compression is e_c. The average unit compressive stress is, therefore, $\frac{1}{2}c$. Since k is the ratio of the distance from the neutral axis to the upper fiber to the total effective depth d, that distance equals kd; the breadth of the beam is b. Therefore

$$\Sigma X = \tfrac{1}{2} c b k d \tag{9}$$

Center of Gravity of Compressive Forces. The center of gravity of compressive forces is sometimes called the *centroid of compression.* It here coincides with the center of gravity of the triangle, which is at one-third the height of the triangle from the upper face. Therefore

$$x = \tfrac{1}{3} k d \tag{10}$$

The ratio of the dimension from the steel to the center of the compressive stress to the dimension d equals j and, therefore, the dimension between the centroids of the tensile and the compressive forces equals jd, which equals $(d-x)$.

Position of the Neutral Axis. According to one of the fundamental laws of mechanics, the sum of the horizontal tensile forces must be equal and opposite to the sum of the compressive forces. Ignoring the very small amount of tension furnished by the concrete below the neutral axis, the tension in the steel equals As equals $pbds$ equals the total compression in the concrete which as stated in Equation (9) equals $\frac{1}{2}cbkd$. Therefore

$$pbds = \tfrac{1}{2} cbkd$$

or

$$ps = \tfrac{1}{2} ck \tag{11}$$

The position of the neutral axis is determined by the value of k, which is a function of the steel ratio p and the ratio of the moduli of elasticities n. We must also eliminate s and c. By definition, c equals $\epsilon_c E_c$ and s equals $\epsilon_s E_s$ and n equals $E_s \div E_c$. Substituting in Equation (11), we have

$$p \epsilon_s E_s = \tfrac{1}{2} \epsilon_c E_c k \tag{12}$$

TABLE XVII

Value of k for Various Values of n and p

(Straight-Line Formulas)

n	p									
	.020	.018	.016	.014	.012	.010	.008	.006	.004	.003
10	.464	.446	.427	.407	.385	.358	.328	.292	.246	.216
12	.493	.476	.457	.436	.412	.385	.353	.314	.266	.235
15	.531	.513	.493	.471	.446	.418	.384	.343	.291	.258
18	.562	.544	.524	.501	.476	.446	.412	.369	.315	.279
20	.580	.562	.542	.519	.493	.463	.428	.384	.328	.292
25	.618	.600	.580	.557	.531	.500	.463	.418	.358	.319
30	.649	.631	.611	.588	.562	.531	.493	.446	.384	.344
40	.698	.679	.659	.637	.611	.579	.542	.493	.428	.384

From the two proportional triangles in Fig. 94, we may write the proportion

$$\frac{\epsilon_c}{kd}=\frac{\epsilon_s}{d-kd} \qquad \text{or } \epsilon_c=\epsilon_s\left(\frac{k}{1-k}\right)$$

Substituting in Equation (12) for the ratio $\frac{E_s}{E_c}$ its value n, and for ϵ_c the value just obtained, we have

$$pn=\tfrac{1}{2}\left(\frac{k^2}{1-k}\right) \qquad (13)$$

Solving this quadratic for k, we have

$$k=\sqrt{2pn+p^2n^2}-pn \qquad (14)$$

Values of Ratio of Moduli of Elasticity. The various values for the ratio of the moduli of elasticity n are discussed in the succeeding paragraphs. The values of k for various values of n and p, have been computed in Table XVII. Eight values have been chosen for n, in conjunction with ten values of p, varying by 0.2 per cent and covering the entire practicable range of p, on the basis of which values k has been worked out in the tabular form. Usually the value of k can be determined directly from Table XVII. By interpolating between two values in Table XVII, any required value within the limits of ordinary practice can be determined

TABLE XVIII

Value of j for Various Values of n and p

(Straight-Line Formulas)

n	p									
	.020	.018	.016	.014	.012	.010	.008	.006	.004	.003
10	.845	.851	.858	.864	.872	.881	.891	.903	.918	.928
12	.836	.841	.848	.855	.863	.872	.882	.895	.911	.922
15	.823	.829	.836	.843	.851	.861	.872	.886	.903	.914
18	.813	.819	.825	.833	.841	.851	.863	.877	.895	.907
20	.807	.813	.819	.827	.836	.846	.857	.872	.891	.903
25	.794	.800	.807	.814	.823	.833	.846	.861	.881	.894
30	.784	.790	.796	.804	.813	.823	.836	.851	.872	.885
40	.767	.774	.780	.788	.796	.807	.819	.836	.857	.872

The dimension jd from the center of the steel to the centroid of the compression in the concrete equals $(d-x)$. Therefore

$$j=\frac{d-x}{d}=\frac{d-\frac{1}{3}kd}{d}=1-\frac{1}{3}k \qquad (15)$$

The corresponding values for j have been computed for the several values of p and n, as shown in Table XVIII.

These several values for k and j which correspond to the various values for p and n are shown in Fig. 95, which is especially useful when the required values of k and j must be obtained by interpolation.

Examples. 1. Assume $n=15$ and $p=.01$; how much are k and j?

Solution. Follow up the vertical line on the diagram for the steel ratio, $p=.010$, to the point where it intersects the k curve for $n=15$; the intersection point is $\frac{9}{10}$ of one of the smallest divisions above the .40 line, as shown on the scale at the left; each small division is .020, and, therefore, the reading is $\frac{9}{10}\times.020=.018$ plus .400 or .418, the value of k. Similarly the .010 p line intersects the j curve for $n=15$ at a point slightly above the .860 line or at .861.

2. Assume $n=16$ and $p=.0082$; how much are k and j?

Solution. One must imagine a vertical line (or perhaps draw one) at $\frac{2}{5}$ of a space between the .0080 and .0085 vertical lines for p. This line would intersect the line for $n=15$ at about .388; and the line for $n=18$ at about .416; one-third of the difference (.028) or .009, added to .388 gives .397, the interpolated value. Although this is sufficiently close for practical purposes, the precise value (.398) may be computed from Equation (27). Similarly the value of j

Theoretically, there are an indefinite number of values of n, the ratio of the moduli of elasticity of the steel and the concrete. The modulus for steel is fairly constant at about 29,000,000 or 30,000,000. The value of the initial modulus for stone concrete varies, according to the quality of the concrete, from 1,500,000 to 3,000,000. An

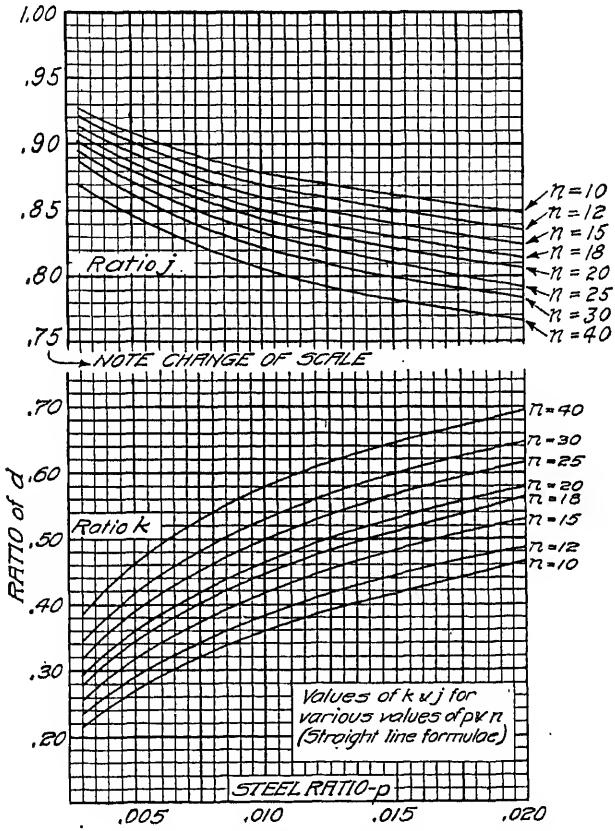


Fig. 95. Curves Giving Values of k and j for Various Values of p and n. Values used for these curves will be found in Tables XVII and XVIII

average value for 1:2:4 cinder concrete is about 1,200,000. Some experimental values for stone concrete have fallen somewhat lower than 1,500,000, while others have reached 4,000,000 and even more. We may use the values in Table XIX with the constant value of 30,000,000 for the steel.

TABLE XIX

Modulus of Elasticity of Some Grades of Concrete

KIND OF CONCRETE	AGE (Days)	MIXTURE	E_c	n
Cinder.	30	1:2:4	1,200,000	25
Broken stone	30	1:3:6	2,000,000	15
Broken stone	10	1:2:4	2,000,000	15
Broken stone	30	1:2:4	2,500,000	12

Percentage of Steel. The previous calculations have been made as if the percentage of the steel might be varied almost indefinitely. While there is considerable freedom of choice, there are limitations beyond which it is useless to pass; and there is always a most economical percentage, depending on the conditions. We must, therefore, determine p in terms of c, s, and n. Substituting in Equation (11) the value of k in Equation (14), we have

$$p=\frac{1}{2}\times\frac{c}{s}\sqrt{2pn+p^2n^2}-\left(\frac{c}{2s}\right)pn$$

which may be reduced to

$$p=\frac{1}{2}\times\frac{c}{s}\times\frac{cn}{(s+cn)} \qquad \textbf{(16)}$$

The above equation shows that we cannot select the percentage of steel at random, since it evidently depends on the selected stresses for the steel and concrete and also on the ratio of their moduli. For example, consider a high-grade concrete—1:2:4—whose modulus of elasticity is considered to be 2,500,000, and which has a working compressive stress c of 600 pounds, which we may consider in conjunction with a tensile stress of 16,000 pounds in the steel. The values of c, s, and n are therefore 600, 16,000, and 12, respectively. Substituting these values in Equation (16), we compute p equals .0058.

The "theoretical" percentage is not, necessarily, the most economical or the most desirable percentage to use. For a beam of given size, some increase of strength may be obtained by using a higher percentage of steel; or for a given strength, or load capacity, the depth may be somewhat decreased by using a higher percentage of steel. The decrease in height, making possible a decrease in the

floors, *may* justify the increase in the percentage of steel, but that is a matter of economics.

Example. What is the theoretical percentage of steel for ordinary stone concrete when $n=15$, $c=650$, and $s=18{,}000$? *Ans.* .0063 per cent

Resisting Moment. The moment which resists the action of the external forces is evidently measured by the product of the distance from the center of gravity of the steel to the centroid of compression of the concrete, times the total compression of the concrete, or times the tension in the steel. As the compression in the concrete and the tension in the steel are equal, it is only a matter of convenience to express this product in terms of the tension in the steel. Therefore, adopting the notation already mentioned, we have the formula

$$M = A\,s\,(jd) \tag{17}$$

But since the computations are frequently made in terms of the dimensions of the concrete and of the percentage of the reinforcing steel, it may be more convenient to write the equation

$$M = (p\,b\,d\,s)\,jd \tag{18}$$

From Equation (9) we have the total compression in the concrete. Multiplying this by the distance from the steel to the centroid of compression $j\,d$, we have another equation for the moment

$$M = \frac{1}{2}(c\,b\,k\,d)\,jd \tag{19}$$

When the percentage of steel used agrees with that computed from Equation (13), then Equations (18) and (19) will give identically the same results; but when the percentage of steel is selected arbitrarily, as is frequently done, then the proposed section should be tested by both equations. When the percentage of steel is larger than that required by Equation (13), the concrete will be compressed more than is intended before the steel attains its normal tension. On the other hand, a lower percentage of steel will require a higher unit tension in the steel before the concrete attains its normal compression. When the discrepancy between the percentage of steel assumed and the true economical value is very great, the stress in the steel, or the concrete, may become dangerously high when the stress in the other element, on which the computation

TABLE XX

Value of p for Various Values of $(s \div c)$ and n

Formula: $p=\frac{1}{2}\times\frac{1}{R}\left(\frac{n}{R+n}\right)$, in which $R=(s\div c)$

$(s \div c)$	n							
	10	12	15	18	20	25	30	40
10	.0250	.0273	.0300	.0321	.0333	.0357	.0375	.0400
12.5	.0178	.0196	.0218	.0236	.0246	.0267	.0282	.0304
15	.0133	.0148	.0167	.0182	.0190	.0208	.0222	.0242
17.5	.0104	.0116	.0132	.0145	.0152	.0168	.0180	.0199
20	.0083	.0094	.0107	.0118	.0125	.0139	.0150	.0167
25	.0057	.0065	.0075	.0084	.0089	.0100	.0109	.0123
30	.0042	.0048	.0056	.0062	.0067	.0076	.0083	.0095
40	.0025	.0029	.0034	.0039	.0042	.0048	.0054	.0062
50	.0017	.0019	.0023	.0026	.0029	.0033	.0037	.0044

Working Values for the Ratio of the Steel Tension to the Concrete Compression. It is often more convenient to obtain working values from tables or diagrams rather than to compute them each time from equations.

Solving Equation (**16**) for several combinations of values of $(s \div c)$ and n, the values are tabulated in Table XX. These values are also shown in Fig. 96. For other combinations than those used in Table XX, the values of p may be obtained with great accuracy provided that $(s \div c)$ corresponds with some curve already on the diagram. If it is necessary to interpolate for some value of $(s \div c)$ of which the curve has not been drawn, it must be recognized that the space between the curves increases rapidly as $(s \div c)$ is smaller. For example, to interpolate for $(s \div c)$ equals 32, the point must be below the 30 curve by considerably more than 0.2 of the interval between the 30 and the 40 curve.

The relative elasticities (n) of various grades of concrete and steel are usually roughly proportional to the relative working values, as expressed by $(s \div c)$. In other words, if n is large, $(s \div c)$ is correspondingly large unless the working value for s or for c is for some reason made abnormally low. Therefore, there will be little if any use for the values given in the lower left-hand and upper right-hand

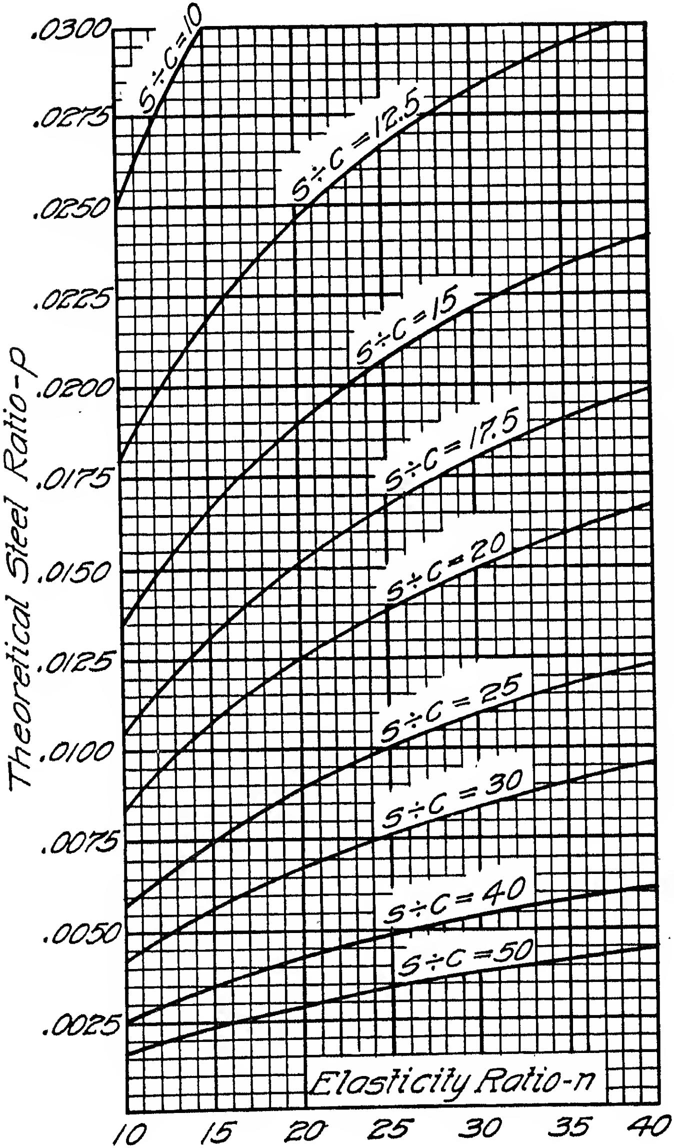


Fig. 96. Curves Showing the Relation of $(s \div c)$ to p and n

Determination of Values for Frequent Use. The moment of resistance of a beam equals the total tension in the steel, or the total compression in the concrete (which are equal), times jd. Therefore, we have the choice of two values, as given in Equations (17) to (19).

$$M_c = \frac{1}{2}(c\,b\,k\,d)\,jd \qquad (20)$$
$$M_s = A\,s\,(jd) = (p\,b\,d\,s)\,jd$$

If the theoretical percentage p has already been determined from Equation (16), then either equation may be used, as most convenient, since they will give identical results. If the percentage has been arbitrarily chosen, then the least value must be determined, as was described on page 189. For any given steel ratio and any one grade of concrete, the factors $\frac{1}{2}ckj$ or psj are constant and Equation (20) may be written

$$M_c = R_c b d^2$$
$$M_s = R_s b d^2$$

or, in general,

$$M = R\,b\,d^2$$

when the theoretical percentage of steel is used. Diagrams for quickly determining R are given in Figs. 99 and 100.

For 1:2:4 concrete, using n equals 15, and with a working value for c equals 600, and s equals 16,000, we find from Equation (16) that the percentage of steel equals

$$p = \frac{1}{2}\,\frac{600}{16{,}000} \times \frac{600 \times 15}{600 \times 15 + 16{,}000} = .00675$$

From Table XVII we find by interpolation that, for n equals 15 and p equals .00675, k equals .360. Then (from Equation (10),

$$x = \frac{1}{3}k\,d = .120\,d \qquad \text{and } j = .880$$

Substituting these values in either formula of Equation (20), we have

$$M = 95\,b\,d^2$$

The percentage of steel computed from Equation (16) has been called *the theoretical percentage*, because it is the percentage which will develop the maximum allowed stress in the concrete and the steel at the same time, or by the loading of the beam to some definite maximum loading. The real meaning of this is best illustrated by

a numerical example using another percentage. Assume that the percentage of steel is exactly doubled, or that p equals $2 \times .00675$ equals .0135. From Table XVII for n equals 15, and p equals .0135, we find k equals .465; x equals $155\,d$; and j equals .845. Substituting these values in both forms of Equation (**20**), we have

$$M_c = 118\,b\,d^2$$
$$M_s = 183\,b\,d^2$$

The interpretation of these two equations, and also of the equation found above ($M = 95\,b\,d^2$), is as follows: Assume a beam of definite dimensions b and d, and made of concrete whose modulus of elasticity is $\frac{1}{15}$ that of the modulus of elasticity of the reinforcing steel; assume that it is reinforced with steel having a cross-sectional area equal to $.00675\,b\,d$. Then, when it is loaded with a load which will develop a moment of $95\,b\,d^2$, the tension in the steel will equal 16,000 pounds per square inch, and the compression in the concrete will equal 600 pounds per square inch at the outer fiber. Assume that the area of the steel is exactly doubled. One effect of this is to lower the neutral axis—k is increased from .360 to .465—and more of the concrete is available for compression. The load may be increased about 24 per cent, or until the moment equals $118\,b\,d^2$, before the compression in the concrete reaches 600 pounds per square inch. Under these conditions the steel has a tension of about 10,340 pounds per square inch, and its full strength is not utilized. If the load were increased until the moment was $183\,b\,d^2$, then the steel would be stressed to 16,000 pounds per square inch, but the concrete would be compressed to about 930 pounds, which would, of course, be unsafe with such a grade of concrete. If the compression in the concrete is to be limited to 600 pounds per square inch, then the load must be limited to that which will give a moment of 118 $b\,d^2$. Even for this the steel is doubled in order to increase the load 24 per cent. Whether this is justifiable, depends on several circumstances—the relative cost of steel and concrete, the possible necessity for keeping the dimensions of the beam within certain limits, etc. Usually a much larger ratio of steel than 0.675 per cent is used; 1.0 per cent is far more common; but when such is used, it means that the strength of the steel cannot be fully utilized unless

indicate higher values of k, which will indicate higher moments; but n cannot be selected at pleasure. It depends on the character of the concrete used; and, with E_s constant, a large value of n means a small value for E_c, which also means a small value for c, the permissible compression stress. Whenever the percentage of steel is greater than the *theoretical* percentage, as is usual, then the upper of the two formulas of Equation (20) should be used. When in doubt, both should be tested, and that one giving the lower moment should be used.

When p equals .0075, n equals 15, c equals 600, and s equals 16,000, as before, we have k equals .374, x equals. 125 d and j equals .875. Then, since p is greater than the theoretical value, we use the upper formula of Equation (20) and have

$$M=98\,b\,d^2$$

Examples. 1. What is the working moment for a slab with 5-inch thickness to the steel, the concrete having the properties described above?

Solution. Let $b=12$ inches, $M=98\times12\times25=29{,}400$ inch-pounds, the permissible moment on a section 12 inches wide.

2. A slab having a span of 8 feet is to support a load of 150 pounds per square foot. The concrete is to be as described above, and the percentage of steel is to be 0.75. What is the required thickness d to the steel?

Solution. Allowing 70 pounds per square foot as the estimated weight of the slab itself, the total load is 220 pounds per square foot. A strip 12 inches wide has an area of 8 square feet, and the total load is 1,760 pounds. Assuming the slab as free-ended, the moment is $\frac{1}{8}\,W\,l=\frac{1}{8}\times1{,}760\times96=21{,}120$ inch-pounds. For a strip 12 inches wide, $b=12$ inches and $M=98\times12\times d^2=1{,}176\,d^2=21{,}120$; from which $d^2=17.96$, and $d=4.24$ inches. Then, allowing one inch of concrete below the steel, the total thickness of the slab would be $5\frac{1}{4}$ inches and its weight, allowing 12 pounds per square foot per inch of depth, would be about 63 pounds per square foot, thus agreeing safely with the estimated allowance for dead load. If the computed thickness and weight had proved to be materially more than the original allowance, another calculation would be necessary, assuming a somewhat greater dead load. This increase of dead load would of itself produce a somewhat greater moment, but the increased thickness would develop a greater resisting moment. A little experience will enable one to make the preliminary estimate so close to the final that not more than one trial calculation should be necessary.

PRACTICAL CALCULATION AND DESIGN OF BEAMS AND SLABS

Tables for Slab Computations. The necessity of very frequently computing the required thickness of slabs renders very

TABLE XXI

Working Loads on Floor Slabs. $M = Wl \div 10$

For $M = Wl \div 8$, subtract 20 per cent from unit loads
For $M = Wl \div 12$, add 20 per cent to unit loads

1 CINDER CONCRETE $c = 300$; $s = 12{,}000$; $n = 25$; $p = .0048$; $M \div bd^2 = 50$

Total Thickness of Slab (Inches)	Thickness of Concrete Below Steel (Inches)	Effective Thickness "d" (Inches)	Area of Steel in 12-inch Width (Sq. In.)	M for 12-inch Width (In.-Lbs.)	Total load in pounds per square foot, including weight of slab, for the given spans in feet												Weight of Slab per Sq. Ft
					4	5	6	7	8	9	10	11	12	13	14	15	
3	3/4	2 1/4	.128	3037	158	101	70	51	39	31	...	...	...	...	...	...	27
3 1/2	3/4	2 3/4	.158	4537	236	151	104	77	59	46	37	...	...	...	...	...	32
4	1	3	.173	5400	281	180	125	91	70	55	45	37	...	...	...	...	36
4 1/2	1	3 1/2	.201	7350	383	245	170	125	95	75	61	50	42	...	...	...	41
5	1	4	.230	9600	500	320	222	163	125	98	80	66	55	47	...	...	45
5 1/2	1	4 1/2	.259	12150	633	405	281	206	158	125	101	83	70	59	51	...	50
6	1 1/4	4 3/4	.273	13537	705	451	313	230	176	139	113	93	78	66	57	...	54
7	1 1/4	5 3/4	.331	19837	...	661	459	337	258	204	165	136	114	97	84	73	63
8	1 1/4	6 3/4	.389	27337	...	911	632	465	356	281	228	188	158	135	116	101	72
9	1 1/2	7 1/2	.432	33750	...	...	781	573	439	347	281	232	195	166	143	125	81
10	1 1/2	8 1/2	.489	43350	...	...	...	737	564	446	361	298	251	213	184	160	90
12	1 1/2	10 1/2	.604	66150	...	...	...	...	861	680	551	456	383	326	281	245	108

2 STONE CONCRETE $c = 500$; $s = 14{,}000$; $n = 15$; $p = .0062$; $M \div bd^2 = 77$

Total Thickness of Slab (Inches)	Thickness of Concrete Below Steel (Inches)	Effective Thickness "d" (Inches)	Area of Steel in 12-inch Width (Sq. In.)	M for 12-inch Width (In.-Lbs.)	4	5	6	7	8	9	10	11	12	13	14	15	Weight of Slab per Sq. Ft
3	3/4	2 1/4	.167	4676	243	156	108	79	60	48	39	...	...	...	...	...	36
3 1/2	3/4	2 3/4	.205	6990	364	233	161	118	91	71	58	48	...	...	...	...	42
4	1	3	.223	8316	433	277	192	141	108	85	69	57	48	...	...	...	48
4 1/2	1	3 1/2	.260	11319	589	377	261	192	147	116	94	78	65	55	...	...	54
5	1	4	.298	14784	769	493	342	251	192	152	123	101	85	72	62	...	60
5 1/2	1	4 1/2	.335	18711	974	623	433	318	243	192	156	129	108	92	79	69	66
6	1 1/4	4 3/4	.353	20846	1085	695	482	354	271	214	174	143	120	102	88	77	72
7	1 1/4	5 3/4	.428	30550	1591	1018	707	519	397	314	254	210	176	150	129	113	84
8	1 1/4	6 3/4	.502	42098	....	1403	974	715	548	437	351	290	243	207	178	156	96
9	1 1/2	7 1/2	.558	51975	....	....	1202	823	677	534	433	351	300	256	216	192	108
10	1 1/2	8 1/2	.632	66759	....	....	1545	1134	868	686	556	460	386	329	283	247	120
12	1 1/2	10 1/2	.780	101871	....	....	....	....	1326	1048	849	702	588	502	433	377	144

TABLE XXI—(Continued)

Working Loads on Floor Slabs. $M = Wl \div 10$

STONE CONCRETE $c = 600$; $s = 16{,}000$; $n = 15$; $p = .00675$; $M \div bd^2 = 95$

For $M = Wl \div 8$, subtract 20 per cent from unit loads
For $M = Wl \div 12$, add 20 per cent to unit loads

Total Thickness Slab (Inches)	Thickness of Concrete Below Steel (Inches)	Effective Thickness "d" (Inches)	Area of Steel in 12-inch Width (Sq. In.)	M for 12-inch Width (In.-Lbs.)	Total load in pounds per square foot, including weight of slab, for the given spans in feet												Weight of Slab per Sq. Ft.
					4	5	6	7	8	9	10	11	12	13	14	15	
3	3/4	2 1/4	.181	5770	300	192	133	98	75	59	48	39	...	...	...	...	36
3 1/2	3/4	2 3/4	.221	8620	449	287	199	146	112	88	72	59	49	...	...	...	42
4	1	3	.241	10260	535	342	238	174	133	105	85	70	59	50	...	...	48
4 1/2	1	3 1/2	.281	13965	727	465	323	237	181	143	116	96	80	68	59	...	54
5	1	4	.321	18240	950	608	422	310	237	187	152	125	105	90	77	67	60
5 1/2	1	4 1/2	.362	23085	1202	769	534	392	300	237	192	159	133	113	98	85	66
6	1 1/4	4 3/4	.382	25720	1340	857	596	437	335	265	214	177	148	127	109	95	72
7	1 1/4	5 3/4	.462	37690	....	1256	872	640	490	388	314	259	218	185	160	139	84
8	1 1/4	6 3/4	.542	51940	....	....	1202	884	676	535	433	357	300	256	221	192	96
9	1 1/2	7 1/2	.603	64125	....	....	....	1090	834	660	534	442	371	316	272	237	108
10	1 1/2	8 1/2	.683	82365	....	....	....	....	1071	847	686	567	476	406	350	305	120
12	1 1/2	10 1/2	.844	125685	....	....	....	....	....	....	1047	866	726	620	534	465	144

STONE CONCRETE $c = 650$; $s = 16{,}000$; $n = 15$; $p = .0077$; $M \div bd^2 = 107$

Total Thickness Slab (Inches)	Thickness of Concrete Below Steel (Inches)	Effective Thickness "d" (Inches)	Area of Steel in 12-inch Width (Sq. In.)	M for 12-inch Width (In.-Lbs.)	4	5	6	7	8	9	10	11	12	13	14	15	Weight of Slab per Sq. Ft.
3	3/4	2 1/4	.208	6490	338	216	150	110	84	66	54	44	37	...	...	...	36
3 1/2	3/4	2 3/4	.254	9710	527	323	224	165	126	99	81	66	56	48	...	...	42
4	1	3	.277	11560	602	385	267	196	150	118	96	79	66	57	49	...	48
4 1/2	1	3 1/2	.328	15730	819	524	364	267	204	162	131	108	91	77	66	58	54
5	1	4	.370	20540	1069	684	475	349	267	211	171	141	118	101	87	76	60
5 1/2	1	4 1/2	.416	26000	...	866	602	442	338	267	216	179	150	128	110	96	66
6	1 1/4	4 3/4	.439	28880	...	962	668	491	376	297	240	199	167	142	122	106	72
7	1 1/4	5 3/4	.531	42450	...	...	982	721	552	436	354	292	245	209	180	157	84
8	1 1/4	6 3/4	.624	58500	...	...	...	994	761	602	487	403	338	288	248	217	96
9	1 1/2	7 1/2	.693	72225	...	...	...	1228	940	743	602	498	418	356	307	267	108
10	1 1/2	8 1/2	.785	92770	...	...	...	...	...	954	773	639	536	457	394	343	120
12	1 1/2	10 1/2	.970	141560	...	...	...	...	...	...	...	975	818	697	602	524	144

TABLE XXI—(Continued)

Working Loads on Floor Slabs. $M = Wl \div 10$

5 STONE CONCRETE $c=650$; $s=18{,}000$; $n=15$; $p=.0063$; $M \div bd^2 = 100$

For $M = Wl \div 8$, subtract 20 per cent from unit loads
For $M = Wl \div 12$, add 20 per cent to unit loads

Total Thickness of Slab (Inches)	Thickness of Concrete Below Steel (Inches)	Effective Thickness "d" (Inches)	Area of Steel in 12-inch Width (Sq. In.)	M for 12-inch Width (In.-Lbs.)	Total load in pounds per square foot, including weight of slab, for the given spans in feet											
					4	5	6	7	8	9	10	11	12	13	14	15
3	¾	2¼	.170	6070	316	202	140	103	79	62	50	41	....	....	....	...
3½	¾	2¾	.208	9075	472	302	210	154	118	93	75	62	52	44	....	...
4	1	3	.227	10800	563	360	250	183	140	111	90	74	62	53	....	...
4½	1	3½	.264	14700	765	490	340	250	191	151	122	101	85	72	62	54
5	1	4	.302	19200	1000	640	444	326	250	197	160	132	111	94	81	71
5½	1	4½	.340	24300	1266	810	562	413	316	250	202	167	140	120	103	90
6	1¼	4¾	.359	27035	1408	901	626	459	351	278	225	186	156	133	115	100
7	1¼	5¾	.434	39675	....	1322	918	674	516	408	330	274	229	195	168	146
8	1¼	6¾	.510	54675	....	....	1265	929	711	563	455	376	316	269	232	202
9	1½	7½	.566	67500	....	....	....	1147	878	694	562	465	390	332	287	250
10	1½	8½	.642	86700	....	....	....	....	1128	892	722	597	501	427	368	321
12	1½	10½	.793	132300	....	....	....	....	....	....	1102	911	764	652	562	490

6 STONE CONCRETE $c=700$; $s=18{,}000$; $n=15$; $p=.0072$; $M \div bd^2 = 113$

Total Thickness of Slab (Inches)	Thickness of Concrete Below Steel (Inches)	Effective Thickness "d" (Inches)	Area of Steel in 12-inch Width (Sq. In.)	M for 12-inch Width (In.-Lbs.)	4	5	6	7	8	9	10	11	12	13	14	15
3	¾	2¼	.194	6850	356	228	158	116	89	70	57	47	39	...	...	...
3½	¾	2¾	.237	10260	534	342	237	174	133	105	85	70	59	50	43	...
4	1	3	.259	12200	635	406	282	207	158	125	101	84	70	60	52	...
4½	1	3½	.302	16610	864	553	385	282	216	170	138	114	96	82	70	61
5	1	4	.346	21700	1130	723	502	369	282	223	181	149	125	107	92	80
5½	1	4½	.389	27460	....	915	636	466	357	282	229	189	158	135	116	101
6	1¼	4¾	.410	30470	....	1015	705	518	396	313	254	210	176	150	129	112
7	1¼	5¾	.496	44830	....	....	1038	762	583	461	373	309	259	221	190	166
8	1¼	6¾	.583	61785	....	....	1430	1050	804	635	515	425	357	304	262	228
9	1½	7½	.648	76275	....	....	....	....	993	784	635	525	441	376	324	282
10	1½	8½	.734	97970	....	....	....	....	...	1007	816	675	567	485	416	362
12	1½	10½	.907	149500	....	....	....	....	..	....	....	1029	865	737	635	553

the basis of several combinations of values of c and s. Municipal building laws frequently specify the unit values which must be used and even the moment formula. For example, slabs are usually continuous over beams and even the wall ends of slabs are so restrained at the wall that the working moment is considerably less than $Wl \div 8$ and, therefore, the formula $Wl \div 10$ is specifically permitted in many municipal regulations. Table XXI is computed on that basis, but the tabulated unit loads may be very easily changed to the basis of $Wl \div 8$ or $Wl \div 12$. It must be noted that the unit loads given in Table XXI include the slab weight, which must, therefore, be subtracted before the net live load is known. In the last column are shown the unit weights of various slab thicknesses on the basis of 108 pounds per cubic foot for cinder concrete and 144 pounds per cubic foot for stone concrete. These subtractive weights may need to be altered if a concrete of different weight is used, or if an extra top coat of concrete, which cannot be considered to be structurally a part of the slab, is laid on afterward. The "thickness of concrete below steel" is such as is approved by good practice, but in case municipal regulations or other reasons should require other thicknesses of concrete below the steel, Table XXI may still be used by considering the *effective* thickness d and by varying, as need be, the subtractive weight of the slab to determine the net load. The blanks in the upper right-hand corner of each section of the table indicate that for those spans and slab thicknesses the slabs cannot safely carry their own weight and that even the weights nearest the blanks are so small that, after subtracting the slab weights, the remainders are too small for practical working floor loads, or even roof loads. The blanks in the lower left-hand corner of each section of the table indicate that for these combinations of span, load, and slab thickness, the shearing strength would be insufficient for the load which its transverse strength would enable it to carry and, therefore, although those slabs would carry a great load, those combinations of span and slab thickness are

pounds net, which is substantially what is required. Another combination would be a 7-inch slab with a span between 10 and 11 feet. To interpolate, subtract 84, the unit slab weight, from 314 and from 259, giving 230 and 175. It should be noted that the difference 388−314, or 74, is greater than the difference 314−259, or 55, which in turn is greater than the difference 259−218, or 41. From this we may know, without precise calculations, that the value for the span 10 feet 6 inches must be such that the difference between 230 (net value) and the net value for 10 feet 6 inches must be greater than the difference between this net value and 175, the net value for an 11-foot span. 230−200=30 and 200−175=25. Therefore, a span of 10 feet 6 inches is very close to the theoretical value—close enough for practical purposes. Whether an 8-inch slab with 12-foot span or a 7-inch slab with 10-foot-6-inch span is most economical or desirable depends on other conditions, one of which is the span of the beams. This will be considered later.

2. Find the span, assuming the same data as above, except that municipal regulations require at least $1\frac{1}{2}$ inches of concrete below the steel and also require using the formula $Wl \div 8$.

Solution. An 8-inch slab with $1\frac{1}{2}$ inches of concrete under the steel will be $8\frac{1}{4}$ inches thick and will weigh 99 pounds per square foot. On the 11-foot span the total load, after subtracting 20 per cent, will be 286 pounds and, after subtracting 99, will leave 187 pounds net. Similarly, the net load on the 10-foot span is 247 pounds. 200−187=13, and 247−187=60; 13 is nearly one-fourth of 60 and, therefore, the interpolated span is about one-fourth of the interval from 11 feet back to 10 feet, or 10 feet 9 inches. The net effect of adding the extra concrete below the steel and using $Wl \div 8$ instead of $Wl \div 10$, therefore, reduces the span of the 8-inch slab from 12 feet to 10 feet 9 inches. A similar computation could be made for a 7-inch slab—actual thickness $7\frac{1}{4}$ inches.

3. Assume a slab made of $1:2\frac{1}{2}:5$ concrete; the span has been determined already as 6 feet; the floor is to be covered with 2 inches of cinder-concrete fill between the wood sleepers and a wood floor, weighing 23 pounds per square foot; the live load is to be 150 pounds per square foot; required the slab thickness.

Solution. For such concrete, use Section 2, Table XXI. 150+23=173, and adding a trial figure of 50 pounds for the unit weight of the slab, we have 223 as the total load. Under 6 feet span we find 192 for a 4-inch slab and 261 for a $4\frac{1}{2}$ inch slab; 4 inches is too thin and $4\frac{1}{2}$ somewhat needlessly thick. Since 223 is nearer to 192 than to 261, we may economize by cutting the thickness to $4\frac{1}{4}$ inches. The detail of the interpolation, elaborated in Example 2, shows this to be justifiable. The required area of steel for the $4\frac{1}{4}$-inch slab is found by interpolation, between .223 and .260, or .242 square inch—the area of steel in 12 inches of width of slab. This is .020 square inch per inch of width; a $\frac{3}{8}$-inch square bar has an area of .1406 square inch; therefore, such bars spaced 7 inches apart will fulfill the requirements.

Practical Methods of Spacing Slab Bars. It is too much to expect of workmen that bars will be accurately spaced when their distance apart is expressed in fractions of an inch. But it is a comparatively simple matter to require the workmen to space the bars evenly, provided it is accurately computed how many bars

TABLE XXII

Gross Load on Rectangular Beam One Inch Wide

For other widths, multiply by width of beam. Based on formula, $M = 100 b d^2$
For any other combination of unit values, multiply by percentage of its formula factor to 100

ctive Depth epth of eam "d" (Inches)	Span in Feet 4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
4	267	213	178	152	133	119	107	97	89	82	76	71	67	....	....	....	....
5	417	333	278	238	208	186	167	151	139	128	119	111	104	98	93	....	....
6	600	480	400	343	300	267	240	218	200	185	171	160	150	141	133	126	120
7	817	653	544	466	408	363	327	297	272	251	233	218	204	192	181	172	163
8	1067	853	711	609	533	474	427	388	356	328	304	284	267	251	237	224	213
9	1350	1080	900	771	675	600	540	491	450	415	385	360	337	317	300	284	270
10	1667	1333	1111	952	833	741	667	606	556	513	476	444	417	392	370	351	333
11	2017	1613	1344	1151	1008	896	807	733	672	620	575	538	504	474	448	424	403
12	2400	1920	1600	1371	1200	1067	960	872	800	738	685	640	600	564	533	505	480
13	2817	2253	1878	1610	1408	1252	1127	1024	939	867	805	751	704	663	626	593	563
14	3267	2613	2178	1866	1633	1452	1307	1188	1089	1005	933	871	817	768	726	687	653
15	3750	3000	2500	2141	1875	1667	1500	1364	1250	1154	1070	1000	937	882	833	789	750
16	4267	3413	2844	2436	2133	1896	1707	1551	1422	1313	1218	1138	1067	1004	948	898	853
17	4817	3853	3211	2752	2408	2141	1927	1751	1606	1482	1376	1284	1204	1133	1070	1014	963
18	5400	4320	3600	3085	2700	2400	2160	1964	1800	1661	1542	1440	1350	1271	1200	1136	1080
19	6017	4813	4011	3437	3008	2674	2407	2188	2006	1852	1718	1604	1504	1416	1337	1266	1203
20	6667	5333	4444	3809	3333	2963	2667	2422	2222	2050	1904	1778	1667	1569	1481	1404	1333

Note. For any beams corresponding to values from the lower left-hand corner of the table, the possible failure by diagonal should be carefully tested.

should be laid in a given width of slab. As an illustration, in Example 3 above, a panel of the flooring, which is, say 20 feet wide, should have a definite number of bars. As 20 feet equals 240 inches and $240 \div 7$ equals 34.3, we shall call this 34, and instruct the workmen to distribute 34 bars equally in the panel 20 feet wide. The workmen can do this without even using a foot-rule, and can adjust the bars to an even spacing with sufficient accuracy for the purpose.

A regulation of the New York City building code is that the spacing of slab bars shall be not greater than $2\frac{1}{2}$ times the thickness of the slab. In the above case the margin is ample; $2\frac{1}{2}$ times $4\frac{1}{4}$ equals 10.6 inches; the designed spacing is 7 inches.

Table for Computation of Simple Beams. In Table XXII has been computed, for convenience, the working total load (including the weight of the beam) on rectangular beams one inch wide and of various depths and spans. For other widths of beams, multiply the tabular load by the width of the beam in inches. Table XXII is based on a grade of concrete such that M equals $100\,b\,d^2$; for any other grade of concrete, determine the corresponding factor of $b\,d^2$, or, in other words, Equation (20), compute the value of $\frac{1}{2}ckj$, or of psj, whichever is least. Multiply the tabular load by the percentage of that factor to 100. The concrete of Section 5, Table XXI, has the factor 100 and if such concrete is used, no percentage multiplication is necessary. The blanks in the upper right-hand corner of Table XXII are similar to the corresponding blanks of the other sections of Table XXI; the beams cannot safely carry their own weight. And, as before, the values immediately adjacent to the blanks are of little or no use, since the possible load, after deducting the weight of the beam, would be too small for practical use. The values in the lower left-hand corner should be used with great caution. Many of the beams of such relative span and depth would fail from diagonal shear long before the tabulated loads were reached. But, since the liability to failure from diagonal shear is dependent on the nature of the web reinforcement, the line of demar-

Solution. From Table XXII, under 18 feet span and opposite 16 inches effective depth, we find 948, the load for a beam one inch wide. An 8-inch beam will carry 8×948, or 7,584 pounds. 95 per cent of 7,584 is 7,205 pounds, the load for that particular grade of concrete. The weight of the concrete, assuming a total depth of 18 inches, is $\frac{8}{12}\times\frac{18}{12}\times18\times144=2{,}592$. Deducting this from 7,205, we have the net load as 4,613 pounds.

2. Assume that $c=500$, $s=16{,}000$, $n=12$, and $p=.006$; how much load will be carried by a beam 6 inches wide, 12 inches effective depth, and 14 feet span?

Solution. From the percentage diagram on page 191, we see that for $s\div c=32$ and $n=12$, $p=.0043$; and since this is less than the chosen steel ratio .006, we must use the first part of Equation (20). For $n=12$ and $p=.006$, $k=.314$ and $j=.895$. Then $\frac{1}{2}ckj=250\times.314\times.895=70$, the factor of bd^2. The load on a beam one inch wide, 12 inches effective depth, and 14 feet span is 685 pounds. For 6 inches wide it would be 4,110 pounds. 70 per cent of this is 2,877 pounds. The weight, allowing 2 inches below the steel, is $\frac{6}{12}\times\frac{14}{12}\times14\times144$, or 1,176 pounds. The net load is, therefore, $4{,}110-1{,}176$, or 2,934 pounds.

BONDING STEEL AND CONCRETE

Resistance to the Slipping of the Steel in the Concrete. The previous discussion has considered merely the tension and compression in the upper and lower sides of the beam. A plain, simple beam resting freely on two end supports has neither tension nor compression in the fibers at the ends of the beam. The horizontal tension and compression, found at or near the center of the beam, entirely disappear by the time the end of the beam is reached. This is done by transferring the tensile stress in the steel at the bottom of the beam to the compression fibers in the top of the beam, by means of the intermediate concrete. This is, in fact, the main use of the concrete in the lower part of the beam.

It is, therefore, necessary that the bond between the concrete and the steel shall be sufficiently great to withstand the tendency to slip. The required strength of this bond is evidently equal to the difference in the tension in the steel per unit of length. For example, suppose that we are considering a bar 1 inch square in the middle of the length of a beam. Let the bar be under an actual tension of 15,000 pounds per square inch. Since the bar is 1 inch square, the actual total tension is 15,000 pounds. Suppose that, at a point 1 inch beyond, the moment in the beam is so reduced that the tension

that the difference of pull (100 pounds) has been taken up by the concrete. The surface of the bar for that length of one inch is four square inches. This will require an average adhesion of 25 pounds per square inch between the steel and the concrete in order to take up this difference of tension. The adhesion between concrete and plain bars is usually considerably greater than this, and there is, therefore, but little question about the bond in the center of the beam. But near the ends of the beam, the change in tension in the bar is far more rapid, and it then becomes questionable whether the bond is sufficient.

Virtue of "Deformed" Bars. The fact that the adhesion of the concrete to the steel is a critical feature under some conditions, called attention to the desirability of using "deformed" bars, which furnish a mechanical bond. Microscopical examination of the surface of steel, and of concrete which has been molded around the steel, shows that the adhesion depends chiefly on the roughness of the steel, and that the cement actually enters into the microscopical indentations in the surface of the metal. Since a stress in the metal even within the elastic limit necessarily reduces its cross section somewhat, the so-called adhesion will be more and more reduced as the stress in the metal becomes greater. This view of the case has been verified by recent experiments by Professor Talbot, who used bars made of tool steel in many of his tests. These bars were exceptionally smooth; and concrete beams reinforced with these bars failed generally on account of the slipping of the bars. Special tests to determine the bond resistance showed that it was far lower than the bond resistance of ordinary plain bars. The designing of the various deformed bars, described on pages 81–83, is only a development of this same principle. The accidental roughness of rolled bars is purposely magnified and the resistance is correspondingly increased. The deformed bars have a variety of shapes; and since they are not prismatic, it is evident that, apart from adhesion, they cannot be drawn through the concrete without splitting or crushing the concrete immediately around the bars. The choice of form is chiefly a matter of designing a form which will furnish the greatest resistance, and which at the same time is not unduly expensive to manufacture. Non-partisan tests have shown that, even under conditions which are most favorable to the plain bars, the deformed

TABLE XXIII

Bond Adhesion of Plain and Deformed Bars per Inch of Length

Basis { 75 lb. adhesion per square inch for plain bars
125 lb. adhesion per square inch for deformed bars }

For any other unit basis, multiply surface (column 2 or 3) by unit

Size of Bar Inches	Surface; (Square Inches per Lineal Inch)		Bond Adhesion per Lineal Inch			
			Plain Bars at 75		Deformed Bars at 125	
	Square	Round	Square	Round	Square	Round
$\frac{1}{4}$	1.00	0.785	75	59	125	98
$\frac{5}{16}$	1.25	0.982	94	74	156	123
$\frac{3}{8}$	1.50	1.178	112	88	187	147
$\frac{7}{16}$	1.75	1.375	131	103	219	172
$\frac{1}{2}$	2.00	1.571	150	118	250	196
$\frac{5}{8}$	2.50	1.964	187	147	312	245
$\frac{3}{4}$	3.00	2.356	225	177	375	294
$\frac{7}{8}$	3.50	2.749	262	206	437	344
1	4.00	3.142	300	236	500	393
$1\frac{1}{8}$	4.50	3.534	337	265	562	442
$1\frac{1}{4}$	5.00	3.927	375	324	625	491

bars have an actual hold in the concrete which is from 50 to 100 per cent greater than that of plain bars. It is unquestionable that age will increase rather than diminish the relative inferiority of plain bars.

The specifications of the American Railway Engineering Association, adopted in 1910, allow 80 pounds per square inch of surface for plain bars, 40 for drawn wire, and from 100 to 150 for deformed bars "depending upon form". Municipal regulations frequently limit the adhesion to 75 pounds, without any mention of deformed bars or of any extra allowable adhesion if such are used. The adhesion is of special importance in short but deep, heavily loaded beams. It is frequently difficult to obtain the necessary adhesion with an allowance of only 75 pounds per square inch. For convenience, Table XXIII is given.

Computation of the Bond Required in Bars. From theoretical mechanics, we learn that the total shear at any section equals the difference in moment for a section of infinitesimal length. This may be seen from Fig. 97 where T is tension in steel at left end of

right end of section; then $T-T'$ is the difference in tension, which is the amount of tension taken up by the concrete in the length x. Then $(T-T')jd$ is the difference of moment in the unit distance x. But by taking moments about a, we have

$$Vx=(T-T')jd$$

from which

$$(T-T')=Vx\div jd$$

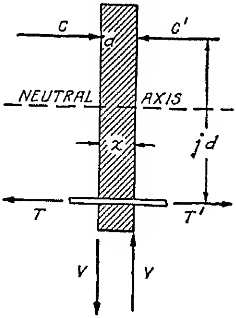


Fig. 97. Diagram for Calculating Moments of Inertia in a Bar

If x is considered to be the unit length —say one inch—then the bond adhesion on *all* the bars will be $V\div jd$. If we call v the unit horizontal shear, and the width of the beam b, then

$$v=V\div bjd \qquad \textbf{(21)}$$

Illustrative Example. Assume an 8-foot beam, uniformly loaded to its capacity, with an effective depth $d=16$ inches, width $b=8$ inches, $c=600$, $s=16{,}000$, and $n=15$. Then $p=.00675$, $k=.360$, $j=.880$, and $A=16\times8\times.0067=0.86$ square inch. This area may be obtained from three $\frac{5}{8}$-inch round bars, each of which will have a cross-sectional area of .30 square inch and circumference of 1.96 inches, which means an adhesion area of 5.88 square inches per inch of length of the three bars. M equals $95bd^2$ or 194,560 inch-pounds equals $Wl\div8$. Since $l=96$ inches, $W=16{,}213$, and V, the maximum total shear, is one-half of this or 8,107 pounds. At a point one foot from the center the shear will be one-fourth of the maximum shear, or 2,027 pounds, and dividing this by jd, or $.880\times16$, we have 144 pounds, the required bond adhesion at that point. Dividing this by the area, 5.88, we have 24 pounds per square inch, the adhesion stress, which is amply safe.

At the abutment the shear is 8,107 pounds; dividing this by jd, or $.880\times16$, we have 575 pounds, the required total adhesion. $575\div5.88$ is 98, the required unit adhesion. This is greater than the permissible unit adhesion of plain bars, and greater than the uniform figure (75) given in so many municipal building codes, although not greater than that which deformed bars can safely carry. Another possible solution of the problem, although at some loss of economy, would be to use four $\frac{1}{2}$-inch square bars, whose total cross-sectional area would be one square inch (instead of 0.86) and whose

superficial area per inch of length would be 8 square inches. $578 \div 8 = 72$ pounds per square inch. This is within the specified limit for plain bars. Strictly speaking, this would not be the precise figure, since the added percentage of steel would slightly decrease j and therefore slightly increase the required adhesion, but the effect in this case is very slight, about one pound per square inch.

Since the variation of j is very little for the usual variations in percentage of steel and quality of concrete, it is a common practice to consider that, *as applied to this equation*, j has the uniform value of .875 or $\frac{7}{8}$. This would reduce Equation (**21**) to

$$v = \frac{8}{7} V \div bd$$

which means that v, the maximum unit horizontal or vertical shear in a section, is about $\frac{1}{7}$ more than the average shear, found by dividing the total shear by the effective section of the beam.

VERTICAL SHEAR AND DIAGONAL TENSION

Distribution of Vertical Shears. Beams which are tested to destruction frequently fail at the ends of the beams, long before the transverse strength at the center has been fully developed. Even if the bond between the steel and the concrete is amply strong for the requirements, the beam may fail on account of the shearing or diagonal stresses in the concrete between the steel and the neutral axis. The student must accept without proof some of the following statements regarding the distribution of the shear.

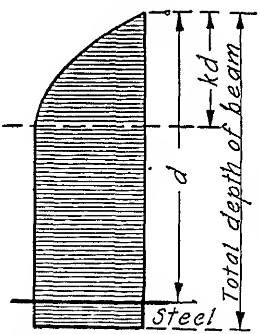


Fig. 98. Diagram Showing Intensity of Shear in Various Points in Height of Beam

The intensity of the shear of various points in the height of the beam may be represented by the diagram in Fig. 98. If we ignore the tension in the concrete due to transverse bending, the shear will be uniform between the steel and the neutral axis. Above the neutral axis, the shear will diminish toward the top of the beam, the curve being parabolic.

The maximum diagonal tensile stress t at any point in a homogeneous beam may be represented by the equation

$$t = \frac{1}{2} f + \sqrt{\frac{1}{4} f^2 + v^2}$$

in which f is the unit horizontal tensile stress and v the unit vertical or horizontal shearing stress. The direction of this maximum tensile stress is given by the formula

$$\tan 2\theta = 2v \div f$$

in which θ is the angle of the maximum tension with the horizontal.

The application of these equations to reinforced concrete beams is uncertain and unreliable, since it depends on assumptions which are themselves uncertain. If there were absolutely no tension in the concrete, f would equal 0, t would equal v, and θ would equal 45°. But there is always some tension in the concrete and this increases t. If there is no web reinforcement, or if *all* the bars run straight through the beam for their entire length, the equations might be used, provided we could know how much tension is actually taken up by the concrete and how much by the steel. According to the best information on the subject, derived from actual tests, t varies from once to twice v, and since v is readily computed from Equation (20), this value may be used as an approximate measure of the *probable* value of t.

Methods of Guarding against Failure by Shear or Diagonal Tension. The failure of a beam by actual shear is almost unknown. The failures usually ascribed to shear are generally caused by diagonal tension. A solution of the very simple Equation (21) will indicate the intensity of the vertical shear. If a beam is so reinforced that it will safely stand the tests for moment, diagonal shear, and bond adhesion, there is almost no question of its ability to resist vertical shear.

Resistance to Diagonal Tension by Bending Bars or Use of Stirrups. Resistance to diagonal tension is furnished by bending up the main reinforcing bars, and also by the use of "stirrups". Unfortunately, it seems impossible to devise any simple, practicable rules (like those for resisting moment) for the precise design of reinforcement to resist diagonal tension.

Professor Talbot (Bulletin No. 29, University of Illinois) suggests that the working stress P in a stirrup may be computed from the formula

$$P = Va \div jd \qquad (22)$$

those previously used. At the same time, he admits that the stress in the stirrup cannot be developed until incipient failure by diagonal tension has already commenced. The rule seems to have the advantage of being amply safe, and since the cost of stirrups is proportionally small, the very slight additional cost of a possible excess in strength is justifiable. Applying the rule to the problem on page 205, the shear at the abutment is 8,107; for a stirrup spacing $a=6$ inches

$$P=(8{,}107\times 6)\div(.881\times 16)=3{,}454 \text{ lb.}$$

Each bar of the stirrup would hold 1,727 pounds, which at 16,000 pounds per square inch would require .11 square inch, which is exactly the area of a $\frac{3}{8}$-inch round bar. But it would be impossible to develop even this tension in the stirrup bars unless they were looped at the top, since they are never long enough to develop a bond adhesion equal to the tensile strength. If the beam is capped by a slab, the stirrup should bend over and extend some distance into the slab.

Resistance to diagonal tension is most efficiently provided by bending up the bars diagonally as fast as they can be spared from their primary work of resisting transverse moment. Diagonal tension tends to produce diagonal cracks which start at the bottom of the beam and develop upward and toward the center. If some of the bars are bent up from the bottom near the ends of the beam, those bars will be nearly normal to these cracks and will resist such tension. From this standpoint alone, it would be preferable to use a large number of small bars, so that a pair of them could be turned up at intervals not greater than the depth of the beam and still have left at least one pair of bars to extend straight through to the end of the beam. But the use and the bending up of a very large number of small bars adds considerably to the cost of small beams, although a large number of bars is sometimes necessary with very large beams. Therefore, although one or two pairs of bars are usually turned up diagonally near the ends of each beam, where the diagonal shear is the greatest, stirrups are depended on to resist diagonal tension.

Example. Assume a plain beam with a span of 18 feet, which is carrying a total load of 1,800 pounds per running foot or 32,400 pounds. Find the reinforcing bars necessary to take care of the diagonal tension and shear.

Assuming the data of Section 3, Table XXI, $M=95\,b\,d^2=874{,}800$. Then $b\,d^2=9{,}208$. If $b=12$, $d^2=767.3$ and $d=27.7$. Then $A=.00675\times 12\times 27.7=2.24$ square inches. This area will be provided by four $\frac{3}{4}$-inch square bars.

Shear. The total equivalent load is 32,400; the maximum shear is one-half, or 16,200. Applying Equation (21), the horizontal shear below the neutral axis equals $v=V\div b\,j\,d$ or $16{,}200\div(12\times.880\times 27.7)=55$ pounds per square inch, which is safe as a unit stress for true shear, but since the diagonal tension *may* be double this, the beam should be reinforced against diagonal shear. Since there are only four main reinforcing bars and since two should be extended straight through without bending up, it leaves only one pair which may be bent up, the bends commencing about two feet from the support at each end.

Stirrups. Transposing Equation (22), we have $a=P\,j\,d\div V$. Talbot's experiments showed that a considerable shearing stress must be developed before the stirrups will begin to take up any stress. Assume that a safe unit shearing stress $v=30$ pounds is developed in the concrete. Then, by inversion of Equation (21), there will be developed a shear of

$$\begin{aligned} V &= v\,b\,j\,d \\ &= 30\times 12\times .88\times 27.7 \\ &= 8{,}775 \text{ lb.} \end{aligned}$$

$16{,}200-8{,}775=7{,}425$, the shear which should be provided for to be taken up by the first stirrup. Assume that the first stirrup is a pair of $\frac{3}{8}$-inch round bars. The area of the bar is 0.11 square inch and at 16,000 pounds per square inch, a pair of the bars can sustain 3,520 pounds, which is one value for P. Then $a=(3{,}520\times.88\times 27.7)\div 7{,}425=11.6$ inches, the *rate* of spacing for the stirrups at the support. Practically, this means that we should place a stirrup about six inches from the support and the next with a spacing of about 12 inches. At the quarter point, the shear is one-half of 16,200 or 8,100 pounds; but since this is less than 8,775 pounds, the available shearing strength of the concrete, there is no need, on the basis assumed, for stirrups even at the quarter points, nor throughout the middle half of the beam. The accuracy of these calculations depends upon uncertain assumptions and the work illustrates the uselessness of precise computations, especially in view of the fact that the very great resistance to diagonal tension provided by the main bent-up bars has been numerically disregarded. The chief use of this method of stirrup calculation is that it indicates a limit beyond which it is useless to pass. Therefore, if we place stirrups made of $\frac{3}{8}$-inch round steel at either end, the first at 6 inches from the support, the others at successively added intervals of 12, 15, 18, and 24 inches, the fourth stirrup will be 6 feet 3 inches from the support. We may feel sure that such stirrups, especially with the added but uncertain aid furnished by the bending-up of the main reinforcing bars, will fully resist all diagonal tension produced by the assumed load.

Although the above method shows how to calculate all the diagonal tension and shear which can be definitely computed, it is becoming common practice to place stirrups along the entire length of the beam. These serve the purpose of furnishing a skeleton to which the other bars may be wired and thus fixed in place, and

also bind the top and bottom of the beam together. This adds a positive but non-computable amount to the strength of the beam.

Calculations by Diagrams of Related Factors. A very large proportion of concrete work is done with a grade of concrete such that we may call the ratio n of the moduli of the steel and the concrete either 12 or 15. The working values of the stresses in the steel and the concrete, s and c, are determined either by public regulation or by the engineer's estimate of the proper values to be used. The diagrams, Figs. 99 and 100, fully cover the whole range of practicable values for steel and for stone concrete. In the previous problems all values have been calculated on the basis of formulas. By means of these diagrams all needed values, on the basis of the other factors, may be read from the diagram with sufficient accuracy for practical work. In addition, the diagrams enable one to note readily the effect of any proposed change in one or more factors.

Illustrative Examples. 1. If a beam, made of concrete such that $n=15$, is to be so loaded that when the stress in the steel (s) is 16,000, the stress in the concrete (c) shall simultaneously be 600, the steel ratio (p) must be .00675. This is found on the diagram, Fig. 99, for $n=15$, by following the line $s=16{,}000$ to its intersection with the line $c=600$. The intersection point, measured on the steel ratio scale at the bottom of the diagram, reads .00675. Also, running horizontally from the intersection point to the scale at the left, we read $R=95$, which is the factor for bd^2 in the moment equation, Equation (20). Incidentally, the corresponding values of k and j for this steel ratio may be obtained, with greater convenience, from this diagram, although they are also obtainable from the more general diagram, Fig. 95.

2. Assume that, for reasons discussed on page 188, it is decided to increase the steel ratio to 1.2 per cent. Following the vertical line for steel ratio equal to .012, we find it intersects the line $c=600$ at a point where $R=114$, but the point is about halfway between the lines $s=10{,}000$ and $s=12{,}000$, indicating that, using that steel ratio, the stress in the steel for a proper stress in the concrete is far less than the usual working stress, and that it would be about 11,000.

than a proper working value.

3. Assume $p = .004$, $c = 600$, and $n = 15$; how much are R and s? R equals 79 and s equals 22,000, which is impracticably

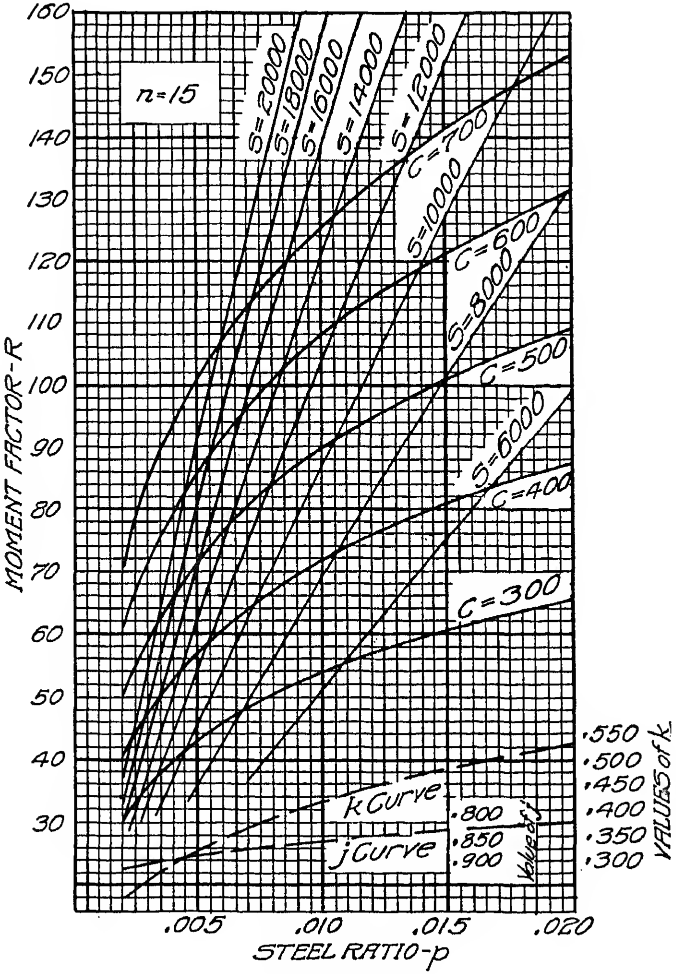


Fig. 99. Curves Showing Values of Moment Factor R for $n = 15$

high. The diagram, Fig. 100, shows plainly that for low steel ratios the values of s are abnormally high for ordinary values of c; on the other hand, for high steel ratios, the ordinary values of c cannot utilize the full working strength of the steel.

especially if very high, is frequently made of steel, even when the floors are made of concrete girders, beams, and slabs. But some-

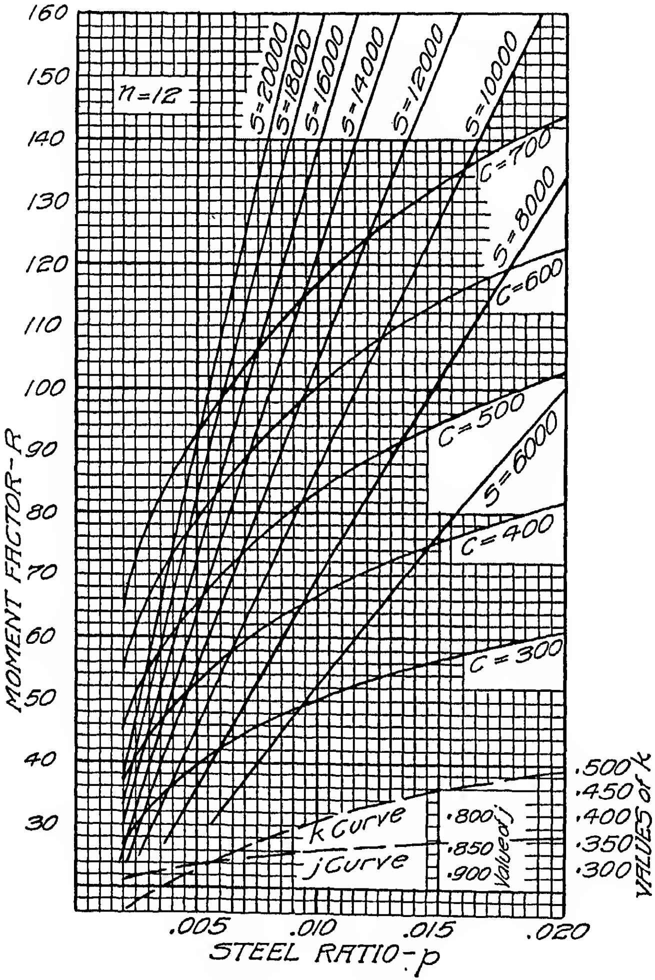


Fig. 100. Curves Showing Values of Moment Factor R for $n=12$

times even the girders and beams are made of steel and only the slab is made of concrete, using steel I-beams for floor girders and beams, and then connecting the beams with concrete floor slabs, Fig. 101. These are usually computed on the basis of transverse beams which

beams, which will add about 50 per cent to their strength. Since it would be necessary to move the reinforcing steel from the lower part to the upper part of the slab when passing over the floor beams, in order to develop the additional strength which is theoretically possible with continuous beams, and since this is not usually done, it is by far the safest practice to consider all floor slabs as being "free-ended". The additional strength, which they undoubtedly have to some extent because they are continuous over the beams, merely adds indefinitely to the factor of safety. Usually, the requirement that the I-beams shall be fireproofed by surrounding the beam itself with a layer of concrete such that the outer surface is at least 2 inches from the nearest point of the steel beam results in having a shoulder of concrete under the end of each slab, which quite mate-

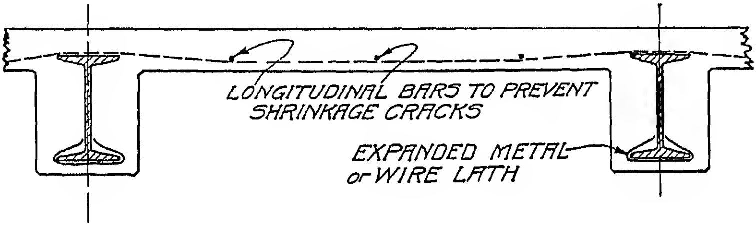


Fig. 101. Diagram Showing Method of Placing Concrete Floor Slabs on I-Beam Girders

rially adds to its structural strength. This justifies the frequent practice of using the moment formula $M = Wl \div 10$, which is a compromise between $Wl \div 8$ and $Wl \div 12$. Even this should only be done when the bars are run into the adjoining span far enough so that the bond adhesion, computed at a safe working value, will not exceed the tension in the steel, and also when the steel is raised to a point near the top of the slab over the supports. The fireproofing around the beam must usually be kept in place by wrapping a small sheet of expanded metal or wire lath around the lower part of the beam before the concrete is placed.

Slabs Reinforced in Both Directions. When the floor beams of a floor are spaced about equally in both directions, so that they form, between the beams, panels which are nearly square, a material saving can be made in the thickness of the slab by reinforcing it with bars running in both directions. The theoretical computation of the

strength of such slabs is exceedingly complicated. The usual method is to estimate that the total load is divided into two parts such that if l equals the length of a rectangular panel and b equals the breadth (l being greater than, or equal to b), then the ratio of the load carried by the "b" bars is given by the proportion $l^4 \div (l^4+b^4)$. If the value of this proportion is worked out for several values of the ratio $l:b$, we have the figures given by the tabular form:

Ratio $l:b$	1.0	1.1	1.2	1.3	1.4	1.5
Proportion of load carried by "b" bars.	50%	59%	67%	74%	80%	83%

When l and b are equal, each set of bars takes half the load. When l is only 50 per cent greater than b, the shorter bars take 83 per cent of the load and it is uneconomical to use bars for transverse moment in the longer direction. The lack of economy begins at about 25 per cent excess length, and therefore panels in which the proportion of length to breadth is greater than 125 per cent should be reinforced in the shorter direction only. Strictly speaking, the slab should be thicker by the thickness of one set of reinforcing bars.

Reinforcement against Temperature Cracks. The modulus of elasticity of ordinary concrete is approximately 2,400,000 pounds per square inch, while its ultimate tensional strength is about 200 pounds per square inch. Therefore, a pull of about $\frac{1}{12000}$ of the length would nearly, if not quite, rupture the concrete. The coefficient of expansion of concrete has been found to be almost identical with that of steel, or .0000065 for each degree Fahrenheit. Therefore, if a block of concrete were held at the ends with absolute rigidity, while its temperature was lowered about 12 degrees, the stress developed in the concrete would be very nearly, if not quite, at the rupture point. Fortunately, the ends will not usually be held with such rigidity; but, nevertheless, it does generally happen that, unless the entire mass of concrete is permitted to expand and contract

prevent the concentration of the stresses at local points, and will distribute it uniformly throughout the mass.

Reinforced-concrete structures are usually provided with bars running in all directions, so that temperature cracks are prevented by the presence of such bars, and it is generally unnecessary to make any special provision against such cracks. The most common exception to this statement occurs in floor slabs, which structurally require bars in only one direction. It is found that cracks parallel with the bars which reinforce the slab will be prevented, if a few bars are laid perpendicularly to the direction of the main reinforcing bars. Usually, $\frac{1}{2}$-inch or $\frac{3}{8}$-inch bars, spaced about 2 feet apart, will be sufficient to prevent such cracks.

Retaining walls, the balustrades of bridges, and other similar structures, which may not need any bars for purely structural reasons, should be provided with such bars in order to prevent temperature cracks. A theoretical determination of the amount of such reinforcing steel is practically impossible, since it depends on assumptions which are themselves very doubtful. It is usually conceded that if there is placed in the concrete an amount of steel whose cross-sectional area equals about $\frac{1}{3}$ of 1 per cent of the area of the concrete, the structure will be proof against such cracks. Fortunately, this amount of steel is so small that any great refinement in its determination is of little importance. Also, since such bars have a value in tying the structure together, and thus adding somewhat to its strength and ability to resist disintegration owing to vibrations, the bars are usually worth what they cost.

T-BEAM CONSTRUCTION

When concrete beams are laid in conjunction with overlying floor slabs, the concrete for both the beams and the slabs being laid in one operation, the strength of such beams is very much greater than their strength considered merely as plain beams, even though we compute the depth of the beams to be equal to the total depth from the bottom of the beam to the top of the slab. An explanation of this added strength may be made as follows:

effective depth to the reinforcement is d. Our previous study in plain beams has shown us that the steel in the bottom of the beam takes care of practically all the tension; that the neutral axis of the beam is somewhat above the center of its height; that the only work of the concrete below the neutral axis is to transfer the stress in the steel to the concrete in the top of the beam; and that even in this work it must be assisted somewhat by stirrups or by bending up the steel bars. If, therefore, we cut out from the lower corners of the beam two rectangles, as shown by the unshaded areas, we are saving a very large part of the concrete, with very little loss in the strength of the beam, provided we can fulfil certain conditions. The steel, instead of being distributed uniformly throughout the bottom of the wide beam, is concentrated into the comparatively narrow portion which we shall hereafter call the rib of the beam. The concentrated tension in the bottom of this *rib* must be transferred to the compression area at the top of the beam. We must also design the beam so that the shearing stresses in the plane mn immediately below the slab shall not exceed the allowable shearing stress in the concrete. We must also provide that failure shall not occur on account of shearing in the vertical planes mr and ns between the sides of the beam and the flanges.

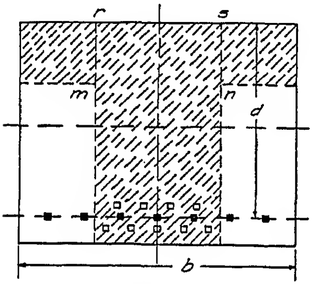


Fig. 102. Diagram of T-Beam in Cross Section

Resisting Moments of T-Beams. The resisting moments of T-beams will be computed in accordance with straight-line formulas. There are three possible cases, according as the neutral axis is: (1) *below* the bottom of the slab (which is the most common case, and which is illustrated in Fig. 103); (2) *at* the bottom of the slab; or (3) *above* it. All possible effect of tension in the concrete is ignored. For Case I, even the compression furnished by the concrete between the neutral axis and the under side of the slab is ignored. Such compression is, of course, zero at the neutral axis; its maximum value at the bottom of the slab is small; the summation of its compression is evidently small; the lever arm is certainly not more than

compared with the resisting moment due to the slab. The computations are much more complicated if it is included; the resulting error is a very small percentage of the true figure, and the error is on the side of safety.

Case I. If c is the maximum compression at the top of the slab, and the stress-strain diagram is rectilinear, as in Fig. 103, then the compression at the bottom of the slab is $c\,\frac{kd-t}{kd}$. The average compression equals $\frac{1}{2}\left(c+c\,\frac{kd-t}{kd}\right)=\frac{c}{kd}\,(kd-\frac{1}{2}t)$. The total compression equals the average compression multiplied by the area $b't$; or

$$C=As=b't\,\frac{c}{kd}\,(kd-\tfrac{1}{2}t) \qquad (23)$$

The center of gravity of the compressive stresses is evidently at the

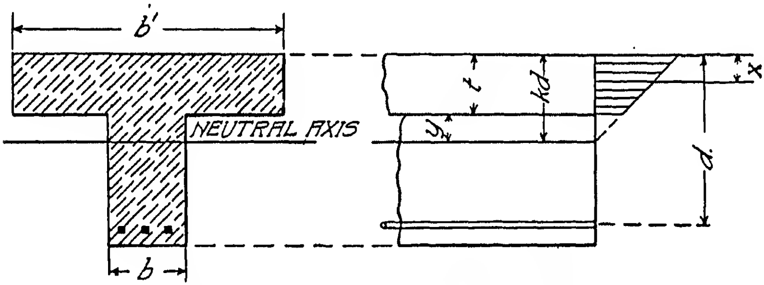


Fig. 103. Compression Stress Diagram for T-Beam

center of gravity of the trapezoid of pressures. The distance x of this center of gravity from the top of the beam is given by the formula

$$x=\frac{t}{3}\times\frac{3kd-2t}{2kd-t} \qquad (24)$$

It has already been shown on page 185 that

$$\frac{\epsilon_c}{\epsilon_s}=\frac{cn}{s}=\frac{kd}{d-kd}$$

Combining this equation with Equation (23), we may eliminate $\frac{c}{s}$ and obtain a value for kd

$$kd=\frac{And+\frac{1}{2}b't^2}{\ldots} \qquad (25)$$

If the percentage of steel is chosen at random, the beam will probably be over-reinforced or under-reinforced. In general it will therefore be necessary to compute the moment with reference to the steel and also with reference to the concrete, and, as before with plain beams (Equation 20), we shall have a pair of equations

$$\left.\begin{aligned} M_c &= C(d-x) = b't\,\frac{c}{kd}(kd-\tfrac{1}{2}t)\,(d-x) \\ M_s &= As(d-x) = pb'ds(d-x) \end{aligned}\right\} \qquad (26)$$

Case II. If we place $kd=t$ in the equation just above Equation (25), and solve for d, we have a relation between d, c, s, n, and t, which holds when the neutral axis is just at the bottom of the slab. The equation becomes

$$d=\frac{t(cn+s)}{cn} \qquad (27)$$

A combination of dimensions and stresses which would place the neutral axis *exactly* in this position is improbable, although readily possible; but Equation (27) is very useful to determine whether a given numerical problem belongs to Case I or Case III. When the stresses s and c in the steel and concrete, the ratio n of the elasticities, and the thickness t of the slab are all determined, then the solution of Equation (27) will give a value of d which would bring the neutral axis at the bottom of the slab. But it should not be forgotten that the compression in the concrete (c) and the tension in the steel will not simultaneously have certain definite values, say $c=500$ and $s=16{,}000$, unless the percentage of steel has been so chosen as to give those simultaneous values. When, as is usual, some other percentage of steel is used, the equation is not strictly applicable, and it therefore should not be used to determine a value of d which will place the neutral axis at the bottom of the slab and thus simplify somewhat the numerical calculations. For example, for $c=500$, $s=16{,}000$, $n=12$, and $t=4$ inches, d will equal 14.67 inches. Of course this particular depth may not satisfy the requirements of the problem. If the proper value for d is *less* than that indicated by Equation (27), the problem belongs to Case III; if it is

having a base c and a height kd which is less than t. The center of compression is at $\frac{1}{3}$ the height from the base, or x equals $\frac{1}{3}\,kd$. Equations (17) to (20) are applicable to this case as well as to Case II, which may be considered merely as the limiting case to Case III. But it should be remembered that b' refers to the width of the flange or slab, and not to the width of the stem or rib.

Width of Flange. The width b' of the flange is usually considered to be equal to the width between adjacent beams, or that it extends from the middle of one panel to the middle of the next. The chief danger in such an assumption lies in the fact that if the beams are very far apart, they must have corresponding strength to carry such a floor load, and the shearing stresses between the rib and the slab will be very great. The method of calculating such shear will be given later. It sometimes happens (as illustrated on page 227), that the width of slab on each side of the rib is almost indefinite. In such a case we must arbitrarily assume some limit. Since the unit shear is greater for short beams than for long beams, the slab thickness should bear some relation to the span of the beam. The building code specifications for New York City limit the width on *each* side of the beam to not greater than one-sixth of the beam span, and not greater than six times the slab thickness. If the width of the rib is twice the slab thickness, this rule permits the width of flange b' to be fourteen times the slab thickness, and something over one-third of the beam span, whichever is least. If the compression is computed for two cases, both of which have the same size of rib, same steel, same thickness of slab, but different slab widths, it is found, as might be expected, that for the narrower slab width the unit compression is greater, the neutral axis is very slightly lower, and even the unit tension in the steel is slightly greater. No demonstration has ever been made to determine any limitation of width of slab beyond which no compression would be developed by the transverse stress in a T-beam rib under it. It is probably safe to assume that it extends for six times the thickness of the slab on *each* side of the rib. If the beam as a whole is safe on this basis, then it is still safer for any additional width to which the compression may extend.

Width of Rib. Since it is assumed that all of the compression

to transfer the tension in the steel to the slab, to resist the shearing and web stresses, and to keep the bars in their proper place. The width of the rib is somewhat determined by the amount of reinforcing steel which must be placed in the rib, and whether it is desirable to use two or more rows of bars instead of merely one row. As indicated in Fig. 102, the amount of steel required in the base of a T-beam is frequently so great that two rows of bars are necessary in order that the bars may have a sufficient spacing between them so that the concrete will not split apart between the bars. Although it would be difficult to develop any rule for the proper spacing between bars without making assumptions which are perhaps doubtful, the following empirical rule is frequently adopted by designers: The *minimum* spacing between bars, center to center, should be two-and-a-quarter times the diameter of the bars. Fire insurance and municipal specifications usually require that there shall be one-and-a-half to two inches clear outside of the steel. This means that the beam shall be three or four inches wider than the net width from out to out of the extreme bars. The data given in Table XXIV will therefore be found very convenient, since, when it is desired to use a certain number of bars of given size, a glance at the table will show immediately whether it is possible to space them in one row; and, if this is not possible, the necessary arrangement can be very readily designed. For example, assume that six $\frac{7}{8}$-inch bars are to be used in a beam. The table shows immediately that, following the rule, the required width of the beam will be 14.72 inches; but if, for any reason, a beam 11 inches wide is considered preferable, the table shows that four $\frac{7}{8}$-inch bars may be placed side by side, leaving two bars to be placed in an upper row. Following the same rule regarding the spacing of the bars in vertical rows, the distance from center to center of the two rows should be $2.25 \times .875$, or 1.97 inches, showing that the rows should be, say two inches apart center to center. It should also be noted that the plane of the center of gravity of this steel is at two-fifths of the distance between the bars above the lower row, or that it is .8 inch above the center of the lower row.

Examples. 1. Assume that a 5-inch slab is supporting a load on beams spaced 5 feet apart, the beams having a span of 20 feet. Assume that the moment

TABLE XXIV

Required Width of Beam, Allowing $2\frac{1}{4} \times d$, for Spacing, Center to Center, and 2 Inches Clear on Each Side

n = number of bars; d = diameter

Formula: $\text{Width} = (n-1)\,2.25d + d + 4 = 2.25\,nd - 1.25d + 4$

No. of Bars	½-in. Bar	⅝-in. Bar	¾-in. Bar	⅞-in. Bar	1-in. Bar	1⅛-in Bar	1¼-in. Bar
	Inches	Inches	Inches	Inches	Inches	Inches	Inches
2	5.62	6.03	6.44	6.84	7.25	7.66	8.06
3	6.75	7.44	8.13	8.81	9.50	8.19	10.87
4	7.87	8.84	9.81	10.78	11.75	12.72	13.68
5	9.00	10.25	11.50	12.75	14.00	15.25	16.50
6	10.12	11.65	13.19	14.72	16.25	17.78	19.31
7	11.25	13.06	14.87	16.68	18.50	20.31	22.12
8	12.37	14.46	16.56	18.65	20.75	22.84	24.94
9	13.50	15.87	18.25	20.62	23.00	25.37	27.75
10	14.62	17.28	19.94	22.59	25.25	27.90	30.56

Note. For side protection of only one and one-half inches, deduct one inch from above figures.

dimensions of the beam if the concrete is not to have a compression greater than 600 pounds per square inch and the tension of the steel is not to be greater than 16,000 pounds per square inch?

Solution. There are an indefinite number of solutions to this problem. There are several terms in Equation (26) which are mutually dependent; it is therefore impracticable to obtain directly the depth of the beam on the basis of assuming the other quantities; therefore, it is only possible to assume figures which experience shows will give approximately accurate results, and then test these figures to see whether all the conditions are satisfied. Within limitations, we may assume the amount of steel to be used, and determine the depth of beam which will satisfy the other conditions, together with that of the assumed area of steel. For example, we shall assume that six ⅞-inch square bars having an area of 4.59 square inches will be a suitable reinforcement for this beam. We shall also assume as a trial figure that x equals 1.5. Substituting these values in the second formula of Equation (26), we may write the second formula

$$900{,}000 = 4.59 \times 16{,}000(d - 1.5)$$

Solving for d, we find that d equals 13.75. If we test this value by means of Equation (27), we shall find that, substituting the values of t, c, n, and s in Equation (27), the resulting value of d equals 16.11. This shows that if we make the depth of the beam only 13.75, the neutral axis will be within the slab, and the problem comes under Case III, to which we must apply Equation (20). Dividing the area of the steel 4.59 by $(b' \times d)$, we have the value of p equals .00556. Interpolating with this value of p in Table XVII, we find that when n equals 12, $k = .303$; $kd = 4.17$; $x = 1.39$; and $jd = 12.36$. Substituting these values in Equation (20), we find that the moment 900,000 equals $1.545\,c$, or that c equals 582 pounds per square inch. This shows that the unit compression of

in the second part of Equation (20), we find that the stress in the steel s equals about 15,860 pounds per square inch.

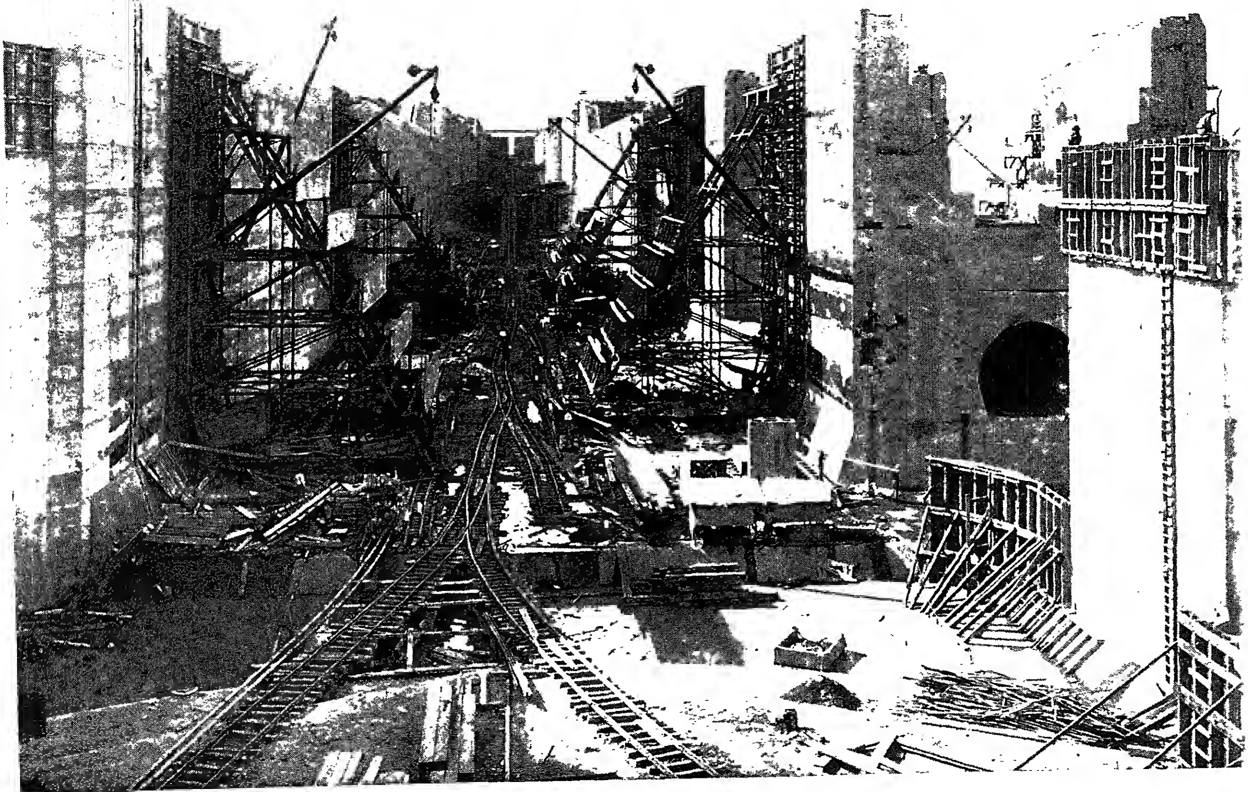
2. Assume that a floor is loaded so that the total weight of live and dead load is 200 pounds per square foot; assume that the T-beams are to be 5 feet apart, and that the slab is to be 4 inches thick; assume that the span of the T-beams is 30 feet. Find the dimensions of the beams.

Solution. We have an area of 150 square feet to be supported by each beam, which gives a total load of 30,000 pounds on each. The moment at the center of such a beam will equal the total load times one-eighth of the span (in inches), or 1,350,000 inch-pounds. As a trial value, we shall assume that the beam is to be reinforced with six $\frac{3}{4}$-inch square bars, which have an area of 3.375 square inches. Substituting this value of the area in the second part of Equation (26), and assuming that s equals 16,000 pounds per square inch, we find as an approximate value for $d-x$, that it will equal 25 inches. This is very much greater than the value of d that would be found from substituting the proper values in Equation (27), so that we know at once that the problem must be solved by the methods of Case I. For a 4-inch slab, the value of x must be somewhere between 1.33 and 2.0. As a trial value, we may call it 1.5, and this means that d will equal 26.5. Assuming that this slab is to be made of concrete using a value for n equal to 12, we know all the values in Equation (25), and may solve for kd, which we find to equal 5.54 inches. As a check on the approximations made above, we may substitute this value of kd, and also the value of t in Equation (24), and obtain a more precise value of x, which we find to equal 1.62. Substituting the value of the moment and the other known quantities in the upper formula of Equation (26), we may solve for the value of c, and obtain the value of 352 pounds per square inch. This value for c is so very moderate that it would probably be economy to assume a lower value for the area of the steel, and increase the unit compression in the concrete; but this solution will not be worked out here.

Calculations by Approximate Formulas. A great deal of T-beam computation is done on the basis that the center of pressure of the concrete is at the middle of the slab and, therefore, that the lever arm of the steel equals $d-\frac{1}{2}t$. From these assumptions we may write the approximate formula

$$M_s = As\,(d-\tfrac{1}{2}t) \tag{28}$$

If the values of M_s and s are known or assumed, we may assume a reasonable value for either A or $d-\frac{1}{2}t$ and calculate the corresponding value of the other. On the assumption that the slab takes all the compression, the distance between the steel and the center of compression of the concrete varies between $d-\frac{1}{2}t$ and $d-.14t$, which is the approximate value when the beam becomes so small that it merges into the slab. The smaller value $d-\frac{1}{2}t$ is the absolute limit which is never reached. Therefore the lever arm is always



GATUN LOWER LOCKS IN PROCESS OF CONSTRUCTION
The view is taken south from construction bridge near north gates.
Courtesy of Panama Canal Commission, United States Government, Washington, D. C.

larger than $d-\frac{1}{2}t$. Therefore, if we use Equation (28) to compute the area of steel A for a definite moment M_s and unit steel tension s, the resulting value of A for an assumed depth d, or the resulting value of d for an assumed area A, will be larger than necessary. In either case the result is safe, but not economically so.

As an illustration, using the values in Example 2, above of $M_s=1{,}350{,}000$; $s=16{,}000$; $(d-\frac{1}{2}t)=26.5-2$ or 24.5, the resulting value of A equals 3.44 square inches, which is larger than the more precise value previously computed.

Equation (28) is particularly applicable when the neutral axis is in the rib. Under this condition, the average pressure on the concrete of the slab is always greater than $\frac{1}{2}c$, or at least it is never less than $\frac{1}{2}c$. As before explained, the average pressure just equals $\frac{1}{2}c$ when the neutral axis is at the bottom of the slab. We may, therefore, say that the total pressure on the slab is always greater than $\frac{1}{2}cb't$. We therefore write the approximate equation

$$M_c=\frac{1}{2}cb't(d-\frac{1}{2}t) \tag{29}$$

As before, the values obtained from this equation are safe, but are unnecessarily so. Applying them to Example 2, by substituting $M_c=1{,}350{,}000$, $b'=60$, $t=4$, and $d-\frac{1}{2}t=24.5$, we compute $c=459$. But we know that this approximate value of c is greater than the true value; and if this value is safe, then the true value is certainly safe. The more accurate value of c, computed in the example cited, is 352. If the value of c in Equation (29) is assumed, and the value of d is computed, the result is a depth of beam unnecessarily great.

If the beam is so shallow that we may know, even without the test of Equation (27), that the neutral axis is certainly within the slab, then we may know that the center of pressure is certainly less than $\frac{1}{3}t$ from the top of the slab, and that the lever arm is certainly less than $d-\frac{1}{3}t$; and we may therefore modify Equation (28) to read

$$M_s=As\,(d-\frac{1}{3}t) \tag{30}$$

Applying this to Example 1, and substituting $M_s=900{,}000$, $s=16{,}000$, $d-\frac{1}{3}t=(13.75-1.67)=12.08$, we find that $A=4.65$, instead of the 4.59 previously computed. This again illustrates that the formula gives an excessively safe value, although in this

the steel and the concrete are safe. It is impracticable to form a simple approximate equation corresponding to Equation (30), which will express the moment as a function of the compression in the concrete. Fortunately it is unnecessary, since, when the neutral axis is within the slab, there is always an abundance of compressive strength.

Shearing Stresses between Beam and Slab. Every solution for T-beam construction should be tested at least to the extent of knowing that there is no danger of failure on account of the shear between the beam and the slab, either on the horizontal plane at the

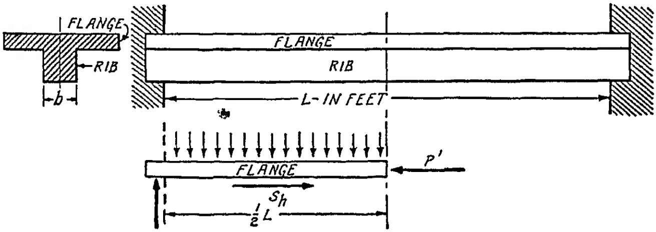


Fig. 104. Diagram Showing Analysis of Stresses in T-Beam

lower edge of the slab, or in the two vertical planes along the two sides of the beam. Let us consider a T-beam such as is illustrated in Fig. 104. In the lower part of the figure is represented one-half of the length of the flange, which is considered to have been separated from the rib. Following the usual method of considering this as a free body in space, acted on by external forces and by such internal forces as are necessary to produce equilibrium, we find that it is acted on at the left end by the abutment reaction, which is a vertical force, and also by a vertical load on top. We may consider P' to represent the summation of all compressive forces acting on the flanges at the center of the beam. In order to produce equilibrium, there must be a shearing force acting on the under side of the flange. We represent this force by S_h. Since these two forces are the only horizontal forces, or forces with horizontal components, which are acting on this free body in space, P' must equal S_h. Let us consider z to

represent the shearing force per unit of area. We know from the laws of mechanics that, with a uniformly distributed load on the beam, the shearing force is maximum at the ends of the beam, and diminishes uniformly towards the center, where it is zero. Therefore the average value of the unit shear for the half length of the beam must equal $\frac{1}{2}z$. As before, we represent the width of the rib by b. For convenience in future computations, we shall consider L to represent the length of the beam, measured in feet. All other dimensions are measured in inches. Therefore the total shearing force along the lower side of the flange will be

$$S_h = \tfrac{1}{2}z \times b \times \tfrac{1}{2}L \times 12 = 3\,b\,z\,L \tag{31}$$

There is also a possibility that a beam may fail in case the flange, or the slab, is too thin; but the slab is always reinforced by bars which are transverse to the beam, and the slab will be placed on both sides of the beam, giving two shearing surfaces.

Numerical Illustration. It is required to test the beam which was computed in Example 1. Here the total compressive stress in the flange equals $\frac{1}{2}cb'kd = \frac{1}{2} \times 582 \times 60 \times 4.17 = 72{,}808$ pounds. But this compressive stress measures the shearing stress S_h between the flange and the rib. This beam requires six $\frac{7}{8}$-inch bars for the reinforcement. We shall assume that the rib is to be 11 inches wide, and that four of the bars are placed in the bottom row, and two bars about 2 inches above them. The effect of this will be to deepen the beam slightly, since d measures the depth of the beam to the center of the reinforcement, and, as already computed numerically on page 220, the center of gravity of this combination will be $\frac{8}{10}$ of an inch above the center of gravity of the lower row of bars. Substituting in Equation (31) the values $S_h = 72{,}808$, $b = 11$, and $L = 20$, we find for the unit value of z 110 pounds per square inch. This shows that the assumed dimensions of the beam are satisfactory in this respect, since the true shearing stress permissible in concrete is higher than this.

But the beam must be tested also for its ability to withstand shear in vertical planes along the sides of the rib. Since the slab in this case is 5 inches thick and we can count on both surfaces to withstand the shear, we have a width of 10 inches to withstand the shear

unit shear would, therefore, be $\frac{11}{10}$ of the unit shear on the underside of the slab, or 121 pounds per square inch. This is at or beyond the limit, 120 pounds, but danger of failure in this respect is avoided by the fact that the slab contains bars which are inserted to reinforce it, and which have such an area that they will effectively prevent any shearing in this way.

Testing Example 2 similarly, we may find the total compression C from Equation (23), which equals $As = 3.375 \times 16{,}000 = 54{,}000$ pounds. The steel reinforcement is six $\frac{3}{4}$-inch bars, and from Table XXIV we find that if placed side by side, the beam must be 13.19 inches in width, or, in round numbers, $13\frac{1}{4}$ inches. $S_h = 54{,}000$, $b = 13.25$, $L = 30$; therefore, from Equation (31), $z = 45$ pounds per square inch. Such a value is of course perfectly safe. The shear along the sides of the beam will be considerably greater, since the slab is only four inches thick, and twice the thickness is but 8 inches; therefore, the maximum unit shear along the sides will equal 45 times the ratio of 13.25 to 8, or 75 pounds per square inch. Even this would be perfectly safe, to say nothing of the additional shearing strength afforded by the slab bars.

Shear in a T-Beam. The shear here referred to is the shear of the beam as a whole on any vertical section. It does not refer to the shearing stresses between the slab and the rib.

The theoretical computation of the shear of a T-beam is a very complicated problem. Fortunately, it is unnecessary to attempt to solve it exactly. The shearing resistance is certainly far greater in the case of a T-beam than in the case of a plain beam of the same width and total depth and loaded with the same total load. Therefore, if the shearing strength is sufficient, according to the rule, for a plain beam, it is certainly sufficient for the T-beam. In Example 1, page 220, the total load on the beam is 30,000 pounds; therefore, the maximum shear V at the end of the beam is 15,000 pounds. In this particular case, jd equals 12.36. For this beam, d equals 13.75 inches and b equals 11 inches. Substituting these values in Equation (22), we have

$$v = \frac{V}{b(jd)} = \frac{15{,}000}{11 \times 12.36} = 113 \text{ lb. per sq. in.}$$

Although this is probably a very safe stress for... it is

more than double the allowable direct tension, 40, due to the diagonal stresses and, therefore, ample reinforcement must be provided. If only two of the $\frac{7}{8}$-inch bars are turned at an angle of 45° at the end, these two bars will have an area of 1.54 square inches, and will have a working tensile strength (at the unit stress of 16,000 pounds) of 24,640 pounds. This is more than the total vertical shear at the ends of the beam, and a pair of turned-up bars would therefore take care of the shear at that point. But considering that stirrups would be used on a beam of 20-foot span, it will be very easy to design these stirrups to provide for this shear, as was explained on page 207.

Numerical Illustration of Slab, Beam, and Girder Construction. Assume a floor construction as outlined in skeleton form in Fig. 105. The columns are spaced 16 feet by 20 feet. Girders which support the alternate rows of beams connect the columns in the 16-foot direction. The live load on the floor is 150 pounds per square foot. The concrete is to be 1:2:4 mixture, with $n=12$ and $c=600$. Required the proper dimensions for the girders, beams, and slab.

Slab. The load on the girders may be computed in either one of two ways, both of which give the same results. We must consider that each beam supports an area of 8 feet by 20 feet. We may therefore consider that girder d supports the load of b (on a floor area 8 feet by 20 feet) as a concentrated load in the center. Or, we may consider that, ignoring the beams, the girder supports a uniformly distributed load on an area 16 feet by 20 feet. The moment in either case is the same. Assume that we shall use a 1 per cent reinforcement in the slab. Then, from Table XVIII, with $n=12$ and $p=.01$, we find that $k=.385$; then $x=.128d$, or $jd=.872d$. As a trial, we estimate that a 5-inch slab (or $d=4$) will carry the load. This will weigh 60 pounds per square foot, and make a total live and dead load of 210 pounds per square foot. A strip one foot wide and 8 feet long will carry a total load of 1,680 pounds, and its moment will be $\frac{1}{8}\times 1{,}680\times 96=20{,}160$ inch-pounds. Using the first half of Equation (20), we can substitute the known values and say that

$$20{,}160=\frac{1}{2}\times 600\times 12\times .385\,d\times .872\,d$$

$$=1{,}209\,d^2$$

$$d^2=16.67$$

In this case the span of the slab is considered as the distance from center to center of the beams. This is evidently more exact than to use the net span—which equals 8 feet, less the still unknown width of beam—since the true span is the distance between the centers of pressure on the two beams. It is probable that the true span (really indeterminable) will be somewhat less than 8 feet, which would probably justify using the round value of $d=4$ inches and the slab thickness as 5 inches, as first assumed. The area of the steel per inch of width of slab equals $pbd=.01\times1\times4.08=.0408$ square inch. Using $\frac{1}{2}$-inch round bars whose area equals .1963 square inch, the required spacing of the bars will be $.1963\div.0408=4.81$ inches. As shown later, the girder will be 11 inches wide and the net *width* of the slab is 240 inches − 11 inches = 229 inches. $229\div4.81=47.6$; call it 48, the number of bars to be spaced equally in one panel. (See page 199.)

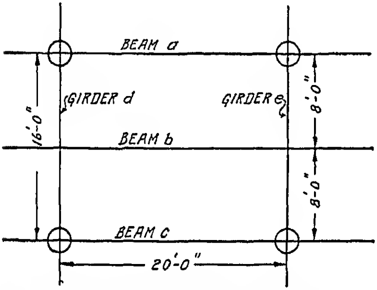


Fig. 105. Skeleton Outline of Floor Panel Showing Slab, Beam, and Girder Construction

Beam. The load on a beam is that on an area of 8 feet by 20 feet, and equals $8\times20\times210$, or 33,600 pounds for live and dead load. As a rough trial value, we shall assume that the beam will be 12 inches wide and 15 inches deep below the slab, or a volume of $1\times1.25\times20$, or 25 cubic feet, which will weigh 3,600 pounds. Adding this, we have 37,200 pounds as the total live and dead load carried by each beam. The load is uniformly distributed and the moment is

$$M=\frac{1}{8}\times37{,}200\times240=1{,}116{,}000 \text{ in.-lb.}$$

We shall assume that the beam is to have a depth d to the reinforcement of 22 inches, and shall utilize Equation (30) to obtain an approximate value for the area. Substituting the known quantities in Equation (30), we have

$$1{,}116{,}000=A\times16{,}000\times(22-1.67)$$

$$A=3.43 \text{ sq. in.}$$

percentage of steel is always very small. In this case, $p=3.43 \div (96 \times 22) = .00162$. Such a value is beyond the range of those given in Table XVII, and therefore we must compute the value of k from Equation (14), and we find $k=.180$ and $kd=3.96$, which shows that the neutral axis is within the slab; $x=\frac{1}{3}kd=1.32$, and therefore $jd=20.68$. Assume that b' equals fourteen times the slab thickness, or 70 inches; see page 219. Substituting these values in the upper part of Equation (20) in order to find the value of c, we find that $c=390$ pounds per square inch. Substituting the known values in the second half of Equation (20), in order to obtain a more precise value of s, we find that $s=15{,}734$ pounds per square inch.

The required area (3.43 square inches) of the bars will be afforded by six $\frac{7}{8}$-inch round bars ($6 \times .60 = 3.60$) with considerable to spare. From Table XXIV we find that six $\frac{7}{8}$-inch bars, either square or round, if placed in one row, would require a beam 14.72 inches wide. This is undesirably wide, and so we shall use two rows, three in each row, and make the beam 9 inches wide. This will add an inch to the depth, and the total depth will be $22+3$, or 25 inches. The concrete below the slab is therefore 9 inches wide by 20 inches deep, instead of 12 inches wide by 15 inches deep, as assumed when computing the dead load, but the weight is the same. It should also be noted that the span of these beams was considered as 20 feet, which is the distance from center to center of the columns (or of the girders). This is certainly more nearly correct than to use the net span between the columns—or girders—which is yet unknown, since neither the columns nor the girders are yet designed. There is probably some margin of safety in using the span as 20 feet.

Girder. The load on one beam is computed above as 37,200 pounds. The load on the girder is, therefore, the equivalent of this load *concentrated* at the center, or of *double* the load (74,400 pounds) uniformly distributed. Assuming for a trial value that the girder will be 12 inches by 22 inches below the slab, its weight for sixteen feet will be 4,224 pounds. This gives a total of 78,624 pounds as the equivalent total live load and dead load uniformly distributed over the girder. Its moment in the center, therefore, equals $\frac{1}{8} \times 78{,}624 \times 192 = 1{,}886{,}976$ inch-pounds.

The width of the slab in this case is almost indefinite, being 20 feet, or forty-eight times the thickness of the slab. We shall

therefore assume that the compression is confined to a width of fourteen times the slab thickness, or that $b'=70$ inches. Assume for a trial value that $d=25$ inches; then from Equation (30), if $s=16{,}000$, we find that $A=5.05$ square inches. Then $p=.00288$; and, from Equation (14), $k=.231$ and $kd=5.775$. This shows that the neutral axis is below the slab, and that it belongs to Case I, page 217. Checking the computation of kd from Equation (25), we compute $kd=5.82$, which is probably the more correct value because computed more directly. The discrepancy is due to the dropping of decimals during the computations. From Equation (24), we compute that $x=1.87$, then $(d-x)=23.13$. Substituting the value of the moment and of the dimensions in the upper part of Equation (26), we compute c to be 409 pounds per square inch. Similarly, making substitutions in the lower part of Equation (26), using the more precise value of $d-x$ for the lever arm of the steel, we find $s=16{,}052$ pounds per square inch. The student should verify in detail all these computations.

The total required area of 5.08 square inches may be divided into, say 8 round bars $\frac{7}{8}$ inch in diameter. These would have an area of 4.81 square inches. The discrepancy is about five per cent. Using the eight round $\frac{7}{8}$-inch bars, the unit stress would be nearly 17,000 pounds. If this is considered undesirable, a more exact area may be obtained by using six $\frac{7}{8}$-inch round bars and two 1-inch round bars. The area would be 5.18 square inches, somewhat in excess of that required. These bars, placed in two rows, would require that the beam should be at least 10.78 inches wide. We shall call it 11 inches. The total depth of the beam will be 3 inches greater than d, or 28 inches. This means 23 inches below the slab, and the area of concrete below the slab is, therefore, 11×23, or 253 square inches, rather than 12×22, or 264 square inches, as assumed for trial.

Shear. The shearing stresses between the rib and slab of the girder are of special importance in this case. The quantity S_h, page 224, equals the total compression in the concrete, which equals the total tension in the steel, which equals, in this case, $16{,}052\times5.08$, or 81,544 pounds. This equals $3bzL$, in which b equals 11, L equals 16 (feet), and z is to be determined.

This measures the maximum shearing stress under the slab and is almost safe, even without the assistance furnished by the stirrups and the bars, which would come up diagonally through the ends of the beam—where this maximum shear occurs—nearly to the top of the slab. The vertical planes on each side of the rib have a combined width of 10 inches, and therefore the *unit stress* is $\frac{11}{10}\times 154$, or 169 pounds per square inch. This is a case of true shear, though it is somewhat larger than the permissible working shear. But there are still other shearing stresses in these vertical planes. Considering a strip of the slab, say one foot wide, which is reinforced by slab bars that are parallel to the girder, the elasticity of such a strip (if disconnected from the girder) would cause it to sag in the center. This must be prevented by the shearing strength of the concrete in the vertical plane along each edge of the girder rib. On account of the combined shearing stresses along these planes, it is usual to specify that when girders are parallel with the slab bars, bars shall be placed across the girder and through the top of the slab for the special purpose of resisting these shearing stresses. Some of the stresses are indefinite, and therefore no precise rules can be computed for the amount of the reinforcement. But since the amount required is evidently very small, no great percentage of accuracy is important. Specifications on this point usually require $\frac{3}{8}$-inch bars, 5 feet long, spaced 12 inches apart.

The shear of the girder, taken as a whole, should be computed as for simple beams, as already discussed on page 226; and stirrups should be used, as described on page 207.

Another special form of shear must be considered in this problem. Where the beams enter the girders there is a tendency for the beams to tear their way out through the girder. The total load on the girder by the two beams on each side is of course equal to the total load on one beam, and equals 37,200 pounds. Some of the reinforcing bars of the beam will be bent up diagonally so that they enter the girder near its top, and therefore the beam could not tear out without shearing through the girder from near its top or for a depth of, say 22 inches (3 inches less than d). If there were no reinforcing steel in the girder and enough load were placed on the beam to actually tear it out, the fracture would evidently be in the

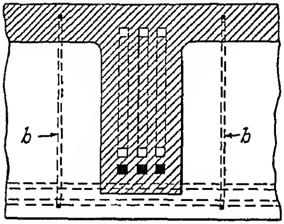


Fig. 106. Details of Reinforcement at Junction of Beam and Girder

and the other dimension, which is the width of the girder rib, 11 inches, there is an area of 484 square inches, and at 40 pounds working tension, it could safely carry a load of 19,360 pounds. But the total load, as shown above, is 37,200 pounds. The steel reinforcement of the girder is, therefore, essential to safety. Although the main reinforcing bars of the girder would have to be torn out before complete failure could take place, the resistance to a small displacement, perpendicular to the bars, is comparatively small, and therefore these bars should not be depended on to resist this stress. But

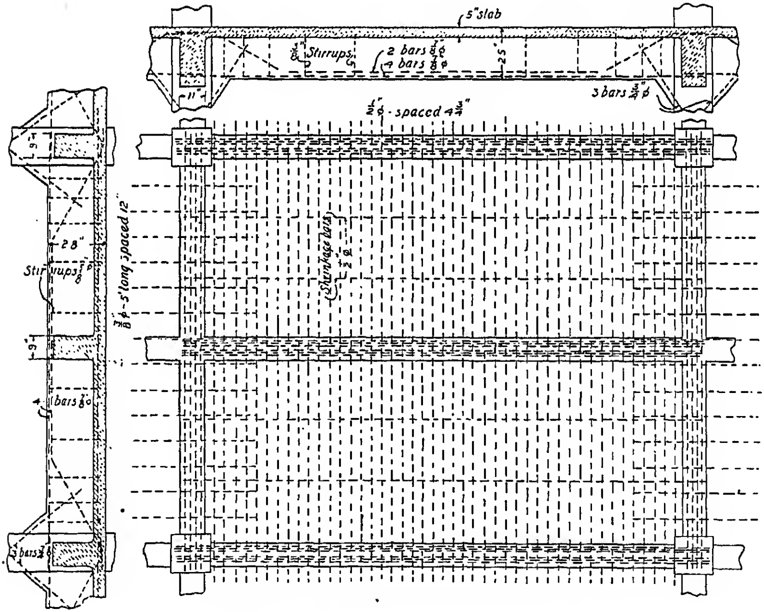


Fig. 107. Detail of Complete Floor Panel

a pair of ordinary vertical stirrups bb, Fig. 106, passing under the main girder bars can easily be made of such size as to take any desired portion, or all, of that load. The stirrups should be bent at

the upper end so that the strength of the bars may be developed without dependence upon bond adhesion. Although precise numerical calculations are impossible without making assumptions which are themselves uncertain, the following calculation is probably safe. $37{,}200-19{,}360=17{,}840$; for s equals 16,000, the required area would be 1.115 square inches. Two pairs of stirrups would give four bar areas which could each be 0.28 square inch, provided by $\frac{5}{8}$-inch round bars. Fig. 107 shows assembled details.

FLAT-SLAB CONSTRUCTION*

Outline of Method. The so-called "flat-slab method" has the advantages that (*a*) there is a very considerable saving in the required height (and cost) of the building on the basis of a given *net* clear height between floors; (*b*) the architectural appearance is improved by having a flat ceiling surface rather than visible beams and girders; (*c*) there is a saving in the cost of forms, not only in surface area and amount of lumber required but also in simplicity of construction, although this saving is offset by an increase in total volume of concrete used; (*d*) there are no deep ceiling beams to cast shadows and it is possible to extend the windows up to the ceiling, which are important items in the lighting of a factory building. Almost the only disadvantage is the difficulty in making perfectly definite and exact computations of the stresses, as may be done for simple beams and slabs. But methods of computation have been devised which, although admittedly approximate, will produce designs for economical construction, and structures so designed have endured, without distress, test loads considerably greater than the designed working loads.

Consider, first, a simple beam, as in Fig. 108-*a*, the beam being continuous over the supports and uniformly loaded for the distance l between the supports with a load amounting to W. Then the maximum moment is located just over the supports and equals $Wl \div 12$. Another local maximum, equal to $Wl \div 24$, is found at the center. Points of inflection are at $.211\,l$ from each column.

Assume that a uniformly loaded plate of indefinite extent is supported on four columns, A, B, C, and D, Fig. 108-*b*, the exten-

plate just over the columns will be horizontal. Then the following conditions may be observed:

(1) The plate will be convex upward over the columns;

(2) the plate will be concave upward at the point O in the center;

(3) there will be curves of inflection, approximately as shown by the dotted curves sketched in around the columns; from the analogy of the simple beam, given above, we may assume that the curves of inflection are approximately at 21 per cent of the span in every direction from the columns.

The columns at the top are made with enlarged sections so as to form a "column head"—which is generally in the form of a frustum of an inverted pyramid or cone, the base being a circle, a square, or a regular polygon.

This device shortens the clear span and decreases the moment. It also increases the size of the hole which the column tends to punch through the plate and hence increases the surface area which resists this punching shear, and thus decreases the unit shear. The diameter of the column head should be about 25 per cent of the span between column centers.

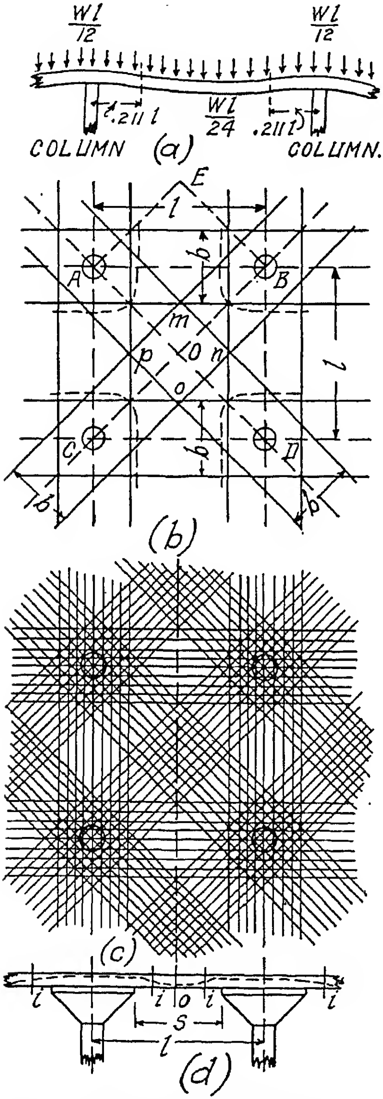


Fig. 108. Diagrams Showing Details of "Flat-Slab" Method of Floor Construction

Placing Reinforcing Bars. Various systems of placing the reinforcing bars have been devised, and some of them patented. The methods may be classified as follows: (1) "Four-way" method, in which the bars run not only in lines parallel to the sides of the rectangles joining the column heads, but also parallel to the diagonals; (2) "two-way" method, in which there are no diagonal bars; and (3) designs which have, in addition to the bands of straight bars from column to column, spirals or a series of rings around the column heads for the specific purpose of providing for the "circumferential tension, or moment". This circumferential tension unquestionably exists, but those who use the first two methods claim that the gridiron of bars formed over the column by the two-way method, and still more so by the four-way method, develops plate action, and that the circumferential stress is amply provided for.

It is a simple matter of geometry to prove that if bands of bars of width b, Fig. 108-b, are placed across columns which form square panels with span l, the width b must equal $.414l$, if the bands exactly cover the space without leaving either gaps or overlaps at m, n, o, and p. The bands may be a little narrower than this, say b equals $.4l$, provided the gaps are not much, if any, greater than the spacing of the bars. On the other hand, the bands should not be wider than twice the diameter of the column head. Fig. 108-c shows that, using the four-way system and with b equal to $.414l$, every part of the slab has at least one layer of bars, some parts have two, some three, and that there are four layers of bars over each column. This is where the moment is maximum.

Method of Calculation. One of the simplest methods of calculation, which probably gives a considerable but undeterminate excess of strength, is to consider the bands as so many simple continuous beams, which are wide but shallow. Consider a *direct* band of width b, equal to $.4l$, the word *direct* being used in contradistinction to *diagonal*. If w is the unit dead and live load per square foot, and s the net span between column heads, then the total load on the band is $.4wls$. Computed as a simple continuous beam, the moment in the center would be $(.4wls)s \div 24$, and that over the columns would be $(.4wls)s \div 12$. By prolonging the steel bars of

adjoining bands sufficiently over a column head so that the bond adhesion is sufficient to develop the full tension over the column head, the total effective area of steel in that band over the column head is double what it is in the center. Practically, this means that the steel should extend to the point of inflection beyond the column head or that its length should be 42 per cent longer than the distance between column centers. Then, on the principle of T-beam flanges, it is assumed that the concrete above the neutral axis for a width of $(b+5t)$ may be computed as taking the compression. For the diagonal bands, the load is $w\times .4l\times 1.414s = .565wls$, and then, considering that a considerable part of the area of the diagonal bands includes that already covered by the direct bands, and also that the diagonal bands both support a square in the center which is one-half of the area lying inside of the direct bands, the moment for the central area is divided between the two diagonal bands and that for each is considered to be $(.565wls\times 1.414s)\div 48 = .0166wls^2$. As before, the moment over the columns for these bands is twice as much, but the steel for the double moment may be obtained, as before, by lapping the bars of adjoining diagonal bands over the columns. The area of a panel, outside of the column heads, which are here assumed to be square, is $l^2-(l-s)^2$. When the column head is 25 per cent of l, then $(l-s)=\frac{1}{4}l$ and the area of the panel is $\frac{15}{16}l^2$, or $.9375l^2$; and the total effective load causing moment on a panel is $W=.9375wl^2$. If we eliminate s and w from the above moment equations, we have

$$\text{Moment at center, direct band} = \frac{(.4wls)s}{24} = \frac{.4wl^3\,\frac{9}{16}}{24} = \frac{3.6}{384}wl^3$$
$$= \frac{Wl}{100}$$

Moment over cap, direct band $= \text{(double the above)} = Wl\div 50$

Moment at center, diagonal band $= .0166wls^2 = Wl\div 100$

Moment over cap, diagonal band $= \text{(double the above)} = Wl\div 50$

Illustrative Example. Assume a live load of 200 pounds per square foot on a square panel 22 feet between column centers. A working rule is that the thickness of the slab should be *at least* $\frac{1}{30}$ of the span; $\frac{1}{30}$ of 22 feet, or 264 inches is 8.8 inches. We will therefore

per square foot. Therefore, $w = 320$ and $W = \frac{15}{16} w l^2 = \frac{15}{16} \times 320 \times 22^2 = 145{,}200$. Then the moment at the center of a direct band equals $W l \div 100 = (145{,}200 \times 264) \div 100 = 383{,}328$ inch-pounds, and the moment for that band over the column is 766,656 inch-pounds. The width of each band b is $.4l = .4 \times 264 = 105.6$ inches. Assume that the steel for one of the bands is placed at 8.5 inches from the compression face, or that d equals 8.5; estimate j equals .91; then

$$
\begin{aligned}
M &= p\,b\,d\,s\,j\,d \\
&= p \times 105.6 \times 8.5 \times 16{,}000 \times .91 \times 8.5 \\
&= 383{,}328
\end{aligned}
$$

from which

$$p = .00345$$

From Table XVIII, we may note that for n equals 15 and p equals .00345, j would be about .91. This checks the assumed value. Then

$$A = p\,b\,d = .00345 \times 105.6 \times 8.5 = 3.10 \text{ sq. in.}$$

This may be amply provided by 13 bars $\frac{1}{2}$ inch square. $105.6 \div 12$, or about 9 inches, gives the spacing of the bars. Although doubling p changes the value of j and will not *exactly* double the moment, yet it will be sufficiently exact to say that double the moment will be obtained over the cap by prolonging the 13 bars of each of the two direct bands in the same line over the columns as far as the circle of inflection, thus doubling the area of the steel. The student should work this out as an exercise. Double p and find the corresponding value of j from Table XVIII; use the actual area of the 26 bars for the value of A, and compute M from $A s j d$. On account of the slight excess in the area of the 26 bars here used, the moment is a little more than necessary.

Location of Bars. There are four layers of bars over the column head and it is evident that they cannot all lie in the same plane or be at the same distance from the compression face. For the layer of bars considered above, d was assumed at 8.5, the maximum permissible with a 10-inch slab. For the next row deduct $\frac{1}{2}$ inch, the

the same, and d is reduced, then p must be increased and j will be less. Assume j equals .90; then

$$\begin{aligned} M &= pbdsjd \\ &= p \times 105.6 \times 8 \times 16{,}000 \times .9 \times 8 \\ &= 383{,}328 \end{aligned}$$

from which

$$p = .00394$$

This is a little more than for the other band, as was expected. Then $A = pbd = 3.33$ square inches, provided by 14 bars $\frac{1}{2}$ inch square. Similarly, it may be shown that reducing d another half-inch for the next layer will add another bar, making 15 bars for the third layer and 16 bars for the fourth layer. Since the computed moments for the direct and diagonal bands is the same for the center of the band, and since the diagonal bands are the longer, there will be some economy in giving them the advantageous position in the slab (larger values of d) and using 13 and 14 bars for the diagonal bands and 15 and 16 bars for the direct bands. The above variation in the number of bars with the change in d indicates the importance of placing the steel exactly as called for by the plans. The design might be made a little more symmetrical, and more foolproof during construction by using 14 bars in each of the diagonal bands and 16 bars in each of the direct bands, and then being sure that the direct bands are *under* the diagonal bands where they pass over the column heads.

Unit Compression. The unit compression may be computed from the equation

$$M = \tfrac{1}{2} c b' k d j d$$

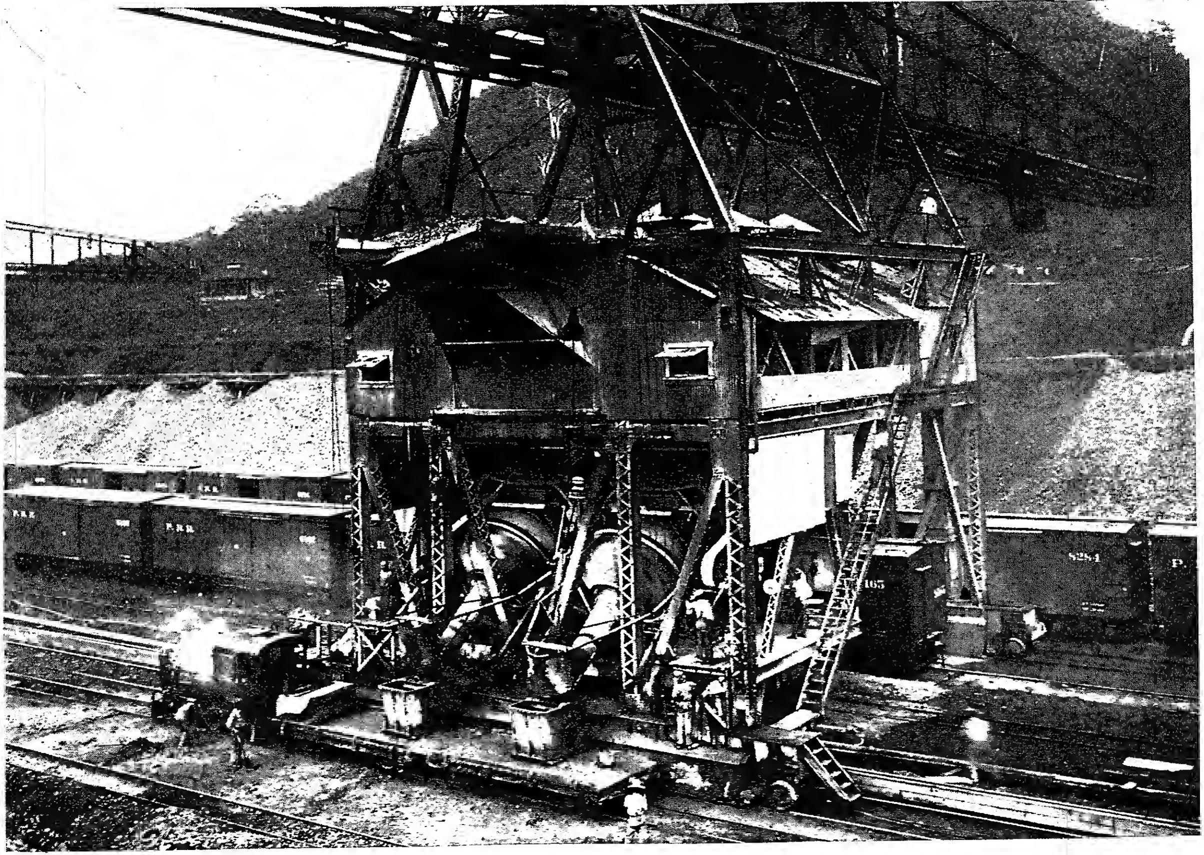
For the concrete compression, we may call $b' = 105.6 + 5t = 105.6 + 50 = 155.6$. The critical place is over the column. Here, where the moment is double,

$$p = A \div b'd = 6.5 \div (155.6 \times 8.5) = .00724$$

Then $M = 766{,}656$; $k = .369$; and $j = .88$.

Substituting these values, we find that

$$c = 420 \text{ pounds per sq. in.}$$



CONCRETE MIXERS LOADING CONCRETE INTO BUCKETS AT PEDRO MIGUEL LOCKS

It will be noted that the whole mixer plant is movable. The steel structure overhead carries crushed stone, etc., to the mixers.

Courtesy of Panama Canal Commission, United States Government, Washington, D. C.

$(155.6 \times 7) = .00734$, which makes $k = .371$ and $j = .88$. Substituting these values, we find that

$$c = 616 \text{ pounds per sq. in.}$$

This is amply safe, especially in view of the fact that a cube subjected to compression on all six faces, as it is in this case, can stand a far higher unit compression than it can when the compression is only on two faces.

Shear. The cap is a square 66 inches on a side and its perimeter is 264 inches. V in this case equals W and is 145,200 pounds. For this calculation let j equal .88 and d equal 8.5; then

$$\frac{V}{bjd} = \frac{145{,}200}{264 \times .88 \times 8.5} = 73.5 \text{ pounds per sq. in.}$$

Since this is a punching shear rather than diagonal tension, this working value is allowable. The usual allowed unit value is 80. At any section farther away from the column head, the total shear is less, and the perimeter, and hence the shearing area, is greater, and therefore the unit shear becomes less and less. The zone around the column head is the critical section and, since it is where the moment is also maximum, no main reinforcing bars can be spared to resist this shear, as is done at the ends of simple beams. A ring of stirrups around each column head is the only practicable method of resisting such shear, if it is excessive.

Wall Panels. The above calculations are virtually for interior panels, or for those where the loads are balanced over the columns. When panels are next to a wall, the bands perpendicular to the wall, and even the diagonal bands, must be anchored by bending them down into the columns. The extra steel is just as necessary, in order to develop the moment at the column head, as if the bands were extended into an adjoining panel. The band along the wall between the wall columns may have part of the usual width cut off. In addition to the floor load, the weight of the wall makes an additional load. This may be most efficiently supported by a "spandrel beam", which is a narrow but deep beam extending up from the floor to the window sill, and which virtually forms that part of the wall, although there may be an outside facing. Sometimes the exterior columns are set in from the building line so as

to partially, if not entirely, balance the load on the other side of the columns.

General Constructive Details. The column head should have a considerable thickness at its edge, immediately under the slab, to enable it to withstand shear, as shown in Fig. 108-*d*. If, as is sometimes done, the sloping sides of the head are continued to the slab surface, a considerable deduction should be made in estimating the effective diameter of the head, which means an increase in the net span between columns. The four points marked *i*, Fig. 108-*d*, are at about 20 per cent of the net span between column heads and are the computed points of inflection where there is no moment. The bars should be in about the middle of the slab at these points. They should be at the minimum permissible distance above the bottom of the slab at *O* and similarly near the top of the slab at the edges and across the column heads. There should not be abrupt bends at these points, but the bars should have easy curves through the required positions at *O* and the points of inflection and then, reversing curvature so that it will be concave downward, should again reach a horizontal direction just over the edge of the column head. While no great precision is essential in locating the bars between these specified points, care must be taken to fasten the bars in exact position at the critical points so that they cannot be disturbed. There should always be at least one inch of concrete below the bars in the center of the slab.

Rectangular Panels. The flat-slab method of construction is most economically used when the panels are nearly, if not quite, square, and also when the column spacing can be made about 23 feet. The ratio of length to breadth for rectangular panels should not exceed 4:3. The two pairs of direct bands must then be computed independently and separately. The diagonal bands must be computed according to their actual dimensions, which means that the moment equations given above will not apply, and other equations, computed in the same general manner, must be derived. The quantity *b* may be considered as 0.4 of the mean of the two column spans. The economy of the flat-slab method is chiefly applicable to heavy floor loadings, such as are required for

REINFORCED-CONCRETE COLUMNS AND WALLS

FLEXURE AND DIRECT STRESS

General Principles. In all of the previous work, the forces acting on a beam are assumed to be perpendicular to the beam; the forces acting on a column are assumed to coincide with the axis of the column. There are many cases in designing in which the resultant of the forces is oblique to the axis of the beam—or column—and, therefore, develops both flexural and direct stress. This is particularly the case in elastic arches. Usually, in concrete work the combination is that of a compressive thrust and flexure, although tension combined with flexure is not impossible. The following demonstration will be made on the basis of the direct stress being exclusively compression.

Columns have reinforcement near two (or four) faces. If the load is eccentric, and especially if it is variable in position, direction, and magnitude, the steel in either face may be alternately in tension and in compression. In the case of arches, steel is placed near the extrados, or upper surface of the arch, and also near the intrados, or lower surface, and variations in the live load may cause the stress in either set of bars to be alternately tension or compression. The reinforcement is, therefore, in compression as well as in tension. And since, for practical reasons, the reinforcement is made uniform throughout the length of the column (beam or arch) and usually the same on both faces, the stresses in the steel are sometimes compression, sometimes tension, sometimes zero, and in general will average far less than the possible safe working value. It is economically impracticable to vary the cross section of the steel to be everywhere at the lowest safe limit of unit stress, especially when the stresses at any section are variable for different loadings. It is, therefore, necessary to use a design which shall be safe for the worst section under the worst condition, although the strength will be excessive at all other sections.

Moment of Inertia of Any Section. In the perfectly general case, the steel near one face is not the same as that near the other. If the steel were replaced by two external "wings" of concrete, each

an area n times the area of the steel ($n = E_s \div E_c$), we would have a section such as is indicated in Fig. 109. O is the "centroid" of that figure, but it is not *necessarily* in the middle of the height.

Let I_c = moment of inertia of the concrete rectangle with respect to the axis through O

I_s = moment of inertia of the areas of steel about the same axis

Then

nI_s = moment of inertia of the concrete wings about the same axis;

I = moment of inertia of the "transformed section"—the rectangle and wings

Then

$$I = I_c + nI_s \tag{32}$$

Let p = steel ratio on tension side (assumed here as lower side) $= (A \div b\,h)$

p' = steel ratio on compression side $= A' \div b\,h$

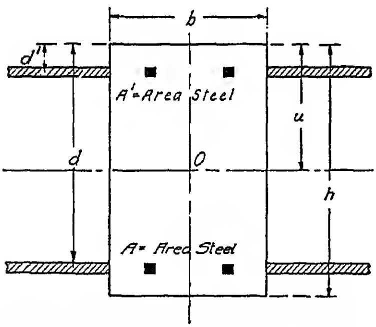


Fig. 109. Diagram Showing Method of Calculating Moment of Inertia of any Section

Then, taking moments about the upper edge of the concrete,

$$u = \frac{b\,h\,(\frac{1}{2}\,h) + n\,A'd' + n\,A\,d}{b\,h + n\,A' + n\,A}$$

But $A' = p'b\,h$ and $A = p\,b\,h$. Then

$$u = \frac{b\,h\,(\frac{1}{2}\,h + n\,p'd' + n\,p\,d)}{b\,h\,(1 + n\,p' + n\,p)}$$

$$u = \frac{\frac{1}{2}\,h + n\,p\,d + n\,p'd'}{1 + n\,p + n\,p'} \tag{33}$$

$$\left.\begin{aligned} I_c &= \tfrac{1}{3}\,b\,[u^3 + (h-u)^3] \\ I_s &= A\,(d-u)^2 + A'\,(u-d')^2 \end{aligned}\right\} \tag{34}$$

When, as is frequently the case, A equals A', and the whole section is, therefore, symmetrical, u equals $\frac{1}{2}\,h$, and the two equations (34) reduce to

$$\left.\begin{aligned} I_c &= \tfrac{1}{12}\,b\,h^3 \\ I_s &= 2A\,(\tfrac{1}{2}\,h - d')^2 \end{aligned}\right\} \tag{35}$$

It is a common practice to make $d' = \frac{1}{10}\,h$, which would make

$$I_s = 2\,A\,(.4\,h)^2 = .32\,A\,h^2$$

Then

of the column, or the tangent to the arch rib. The perpendicular component produces *shear*, and, although it should be tested on general principles to be sure that the section can stand it, it is generally true that the obliquity of the force is so small that the shearing component does not produce a dangerous shearing stress even for plain concrete. The component parallel to the axis is called the *thrust*. Its effect on the section depends on its *eccentricity*, or its distance e from the center of gravity of the section. There are three general cases:

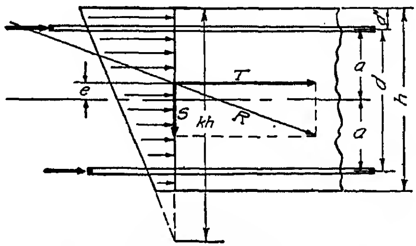


Fig. 110. Diagram Showing Effect of Oblique Force Acting on a Section

First, when e is so small that there is compression over the entire section. When e is 0, the compression is uniform; for very small values of e the compression varies about as shown in Fig. 110, the greatest unit compression being on the side of the eccentric force.

Second, for some special value of e (called e_0); in this case the compression at one face becomes just zero.

Third, for still larger values of e; in this case the stress on the side away from the force T becomes tension. When this tension is still small and less than the unit tension which may safely be sustained by concrete, certain formulas apply. When the eccentricity, and the consequent tension, becomes too great and the tension must all be taken by the steel, other formulas must be used. For simplicity, all of the following demonstrations on this subject will be based on the two very common conditions that the section is rectangular and that the steel reinforcement is the same on both sides.

These cases may now be considered in greater detail under four heads, the first one being divided into two, when $e=0$ and when $e>0$ but still small.

Case I. $e=0$. Then the unit compression in the concrete equals

$$c=\frac{T}{bh}\left(\frac{1}{(1+2np)}\right) \tag{37}$$

and the unit compression in the steel equals

$$s=\frac{nT}{bh}\left(\frac{1}{(1+2np)}\right) \tag{38}$$

Case II. $e>0$, but is so small that there is compression over the entire section. Then the maximum unit compression in the concrete is

$$c=\frac{T}{bh}\left(\frac{1}{1+2np}+\frac{6he}{h^2+24npa^2}\right) \tag{39}$$

and the maximum unit compression in the steel

$$s=\frac{nT}{bh}\left(\frac{1}{1+2np}+\frac{12ae}{h^2+24npa^2}\right) \tag{40}$$

In this case the force T may be considered as replaced by the series of forces shown in Fig. 110—two concentrated forces carried by the steel near top and bottom and a graded series of compressions on the concrete. The minimum unit values of the compression are of little practical importance.

Case III. $e=e_0$, the special value of e, determined later, which will make the compression in the concrete at one face just zero. The maximum unit compression in the concrete equals

$$c=\frac{2T}{bh}\left(\frac{1}{1+2np}\right) \tag{41}$$

which is just twice the value found in Equation (37), which was to be expected. Since Equation (39) is applicable to all values of e between 0 and e_0, we may place the two values of c from Equations (39) and (41) equal and find the value of e, which is the special value e_0

$$e_0=\frac{h^2+24npa^2}{6h(1+2np)} \tag{42}$$

Using this value of e_0 for the e of Equation (40), the unit steel compression is

$$s=\frac{nT}{bh\,(1+2np)}\left(1+\frac{2a}{h}\right) \tag{43}$$

As before, the minimum unit stresses are of no practical importance.

Illustrative Example. Assume a concrete section bh equal to 12 inches by 18 inches, with $\frac{7}{8}$-inch square bars, spaced 6 inches, at top

with the axis of 62,500 pounds is applied 3 inches (e) from the center; required the maximum unit stresses in the concrete and in the steel. Then, since $p = A \div b\,h$,

$$p = \frac{2 \times .7656}{12 \times 18} = .007$$

From Equation (42), if $n = 15$,

$$e_0 = \frac{324 + (24 \times 15 \times .007 \times 51.84)}{6 \times 18\ (1 + 30 \times .007)} = 3.48$$

This being greater than $e = 3$ inches, it shows that the stress is wholly compressive. For this case, and for all cases when n equals 15 and a equals $.4\,h$, we may simplify Equations (39) and (40) to the following:

$$c = \frac{T}{b\,h}\left(\frac{1}{1+2np} + \frac{e}{h}\left(\frac{6}{1+57.6p}\right)\right) \qquad (44)$$

$$s = \frac{n\,T}{b\,h}\left(\frac{1}{1+2np} + \frac{e}{h}\left(\frac{4.8}{1+57.6p}\right)\right) \qquad (45)$$

Then

$$c = \frac{62{,}500}{216}\left(\frac{1}{1+.210} + \frac{3}{18}\left(\frac{6}{1+.4032}\right)\right) = 445 \text{ lb. per sq. in.}$$

and

$$s = \frac{15 \times 62{,}500}{216}\left(\frac{1}{1+.210} + \frac{3}{18}\left(\frac{4.8}{1+.4032}\right)\right) = 6{,}062 \text{ lb. per sq. in.}$$

Case IV. As e is so great there is tension on one face. When e is but little more than e_0, the tension is not greater than the concrete can withstand without rupture and the stresses in both concrete and steel may be determined by equations similar to those given above. But when the tension is evidently so great that the concrete will be ruptured on the tension side, the steel must be considered as carrying *all* the tension and then other formulas must be used, as developed below.

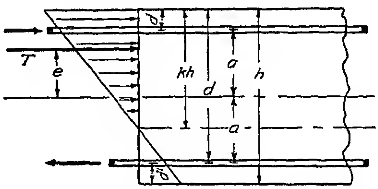


Fig. 111. Diagram Showing Effect of Oblique Force Acting on a Section when Eccentricity e is Large

In Fig. 111 the triangle of forces may be considered as representing proportionately, the deformation in the concrete and also in

certain deformation in steel as would be required with concrete, we may consider that the triangle represents the proportionate stresses in the concrete at the several points in the section and also that the stress in the steel is represented at the same scale by n times the ordinate at the position of the steel, or that the actual ordinate represents $s \div n$. From proportionate triangles we can write

$$\frac{s'}{n}=c\left(\frac{kh-d'}{kh}\right) \qquad \text{or} \qquad s'=nc\left(\frac{kh-d'}{kh}\right) \tag{46}$$

also

$$\frac{s}{n}=c\left(\frac{d-kh}{kh}\right) \qquad \text{or} \qquad s=nc\left(\frac{d-kh}{kh}\right) \tag{47}$$

The algebraic sum of all the forces acting on the section must equal the thrust T. Therefore

$$T=s'pbh+\tfrac{1}{2}cbkh-spbh \tag{48}$$

Substituting the above values for s' and s, we have, after reducing,

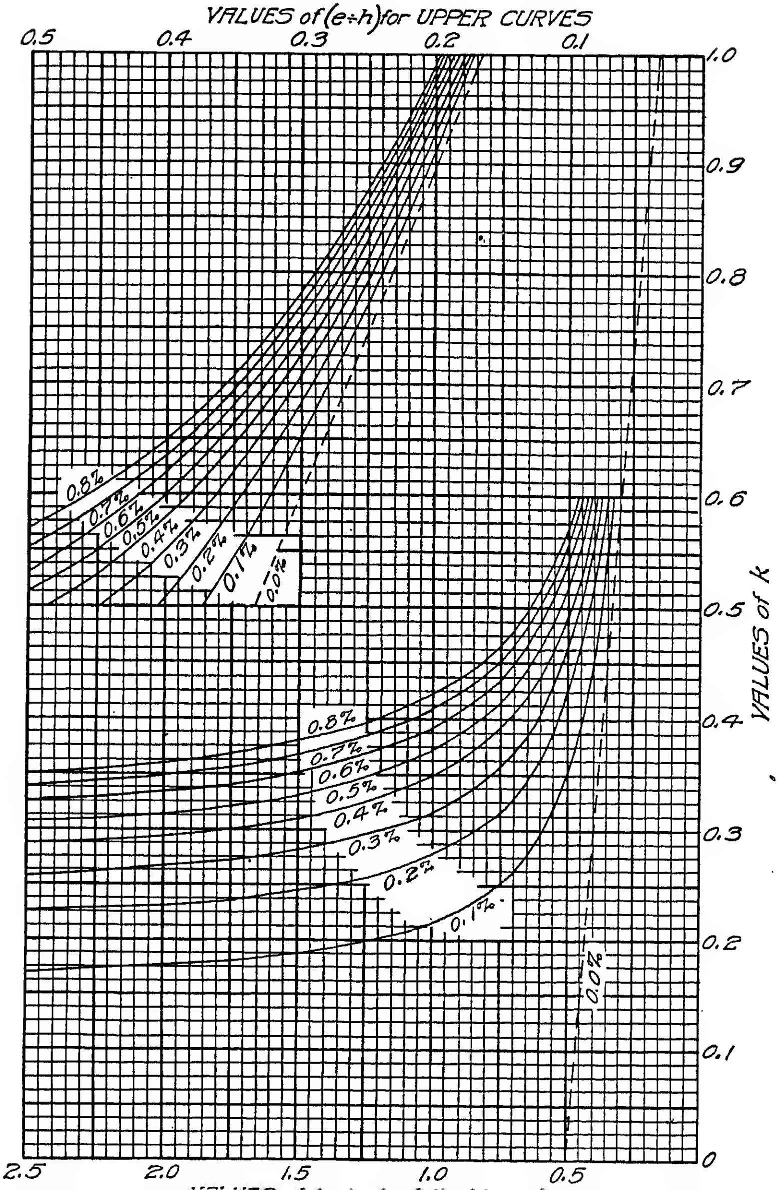
$$T=\frac{cbh}{2}\left(\frac{k^2+4pnk-2pn}{k}\right) \tag{49}$$

But the moment M on this section about the gravity axis evidently equals Te. We may also say that the moment M equals the sum of the moments of the separate forces about the gravity axis. The compressive forces have their center of gravity at one-third the height of the triangle and its distance from the gravity axis is $\frac{1}{2}h-\frac{1}{3}kh$, and the summation of the compressive moments of the concrete equals $\frac{1}{2}cbkh\,(\frac{1}{2}h-\frac{1}{3}kh)$. The entire moment equals

$$\begin{aligned}
M&=\tfrac{1}{2}cbkh(\tfrac{1}{2}h-\tfrac{1}{3}kh)+s'pbha+spbha\\
&=\tfrac{1}{4}cbkh^2-\tfrac{1}{6}cbk^2h^2+nc\left(\frac{kh-d'}{kh}\right)pbha+nc\frac{d-kh}{kh}pbha\\
&=\tfrac{1}{4}(cbkh^2)-\tfrac{1}{6}(cbk^2h^2)+ncpb\,\frac{a}{k}\,(d-d')\\
&=cbh^2\left(\frac{k}{4}-\frac{k^2}{6}+\frac{np2a^2}{kh^2}\right) \qquad (50)
\end{aligned}$$

Placing this equal to the above value for T in Equation (49), multiplied by e, we have, after reduction,

$$k^3+3k^2\left(\frac{e}{h}-\frac{1}{2}\right)+12pnk\frac{e}{h}=12\frac{pna^2}{h^2}+6pn\frac{e}{h} \tag{51}$$


VALUES of (e÷h) for UPPER CURVES
0.5
0.4
0.3
0.2
0.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
VALUES of k
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
0.2%
0.1%
0.0%
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
0.2%
0.1%
0.0%
2.5
2.0
1.5
1.0
0.5

$$k^3+3k^2\left(\frac{e}{h}-\frac{1}{2}\right)+180pk\frac{e}{h}=28.8p+90p\frac{e}{h} \qquad (52)$$

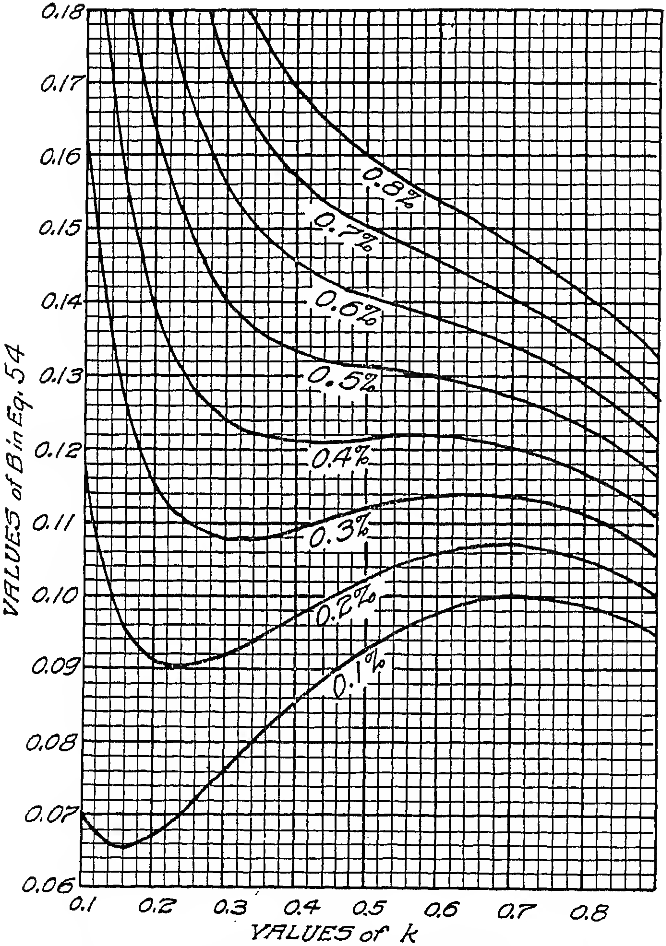


Fig. 113. Relations of k, p, and B for Flexure and Direct Stress

The direct solution of this cubic equation is not easy, but the desired relation between k, p, and $(e \div h)$ may be obtained by assuming all pairs of values for k and p within any desired range, computing the

shown in the diagram, Fig. 112. Then, for any selected values of p and $(e \div h)$, the value of k may be read from the diagram with practicable accuracy.

The practical application of Equation (50) usually consists in the numerical determination of c, on the basis of a beam of given dimensions $(b\,h)$ and with other known characteristics (k, n, p, and a), which is acted on by a known moment M. The value of k is determined from Equation (51) or (52), or by the use of the diagram. But the work can be still further simplified by using another diagram, Fig. 113, for the determination of the value of the parenthesis $\left(\frac{k}{4}-\frac{k^2}{6}+\frac{n\,p\,2a^2}{k\,h^2}\right)$, which we will call equal to B. Then we have

$$c=\frac{M}{b\,h^2 B} \tag{53}$$

As before, using the special values of $n=15$ and $a=.4\,h$, we have

$$B=\frac{k}{4}-\frac{k^2}{6}+\frac{4.8\,p}{k} \tag{54}$$

Numerical examples of this will be given under "Arches", Part V.

FOOTINGS

Simple Footings. When a definite load, such as a weight carried by a column or wall, is to be supported on a subsoil whose bearing power has been estimated at some definite figure, the required area of the footing becomes a perfectly definite quantity, regardless of the method of construction of the footing. But with the area of the footing once determined, it is possible to effect considerable economy in the construction of the footing by the use of reinforced concrete. An ordinary footing of masonry is usually made in a pyramidal form, although the sides will be stepped off, instead of being made sloping. It may be approximately stated that the depth of the footing below the base of the column or wall, when ordinary masonry is used, must be practically equal to the width of the footing. The offsets in the masonry cannot ordinarily be made any greater than the heights of the various steps. Such a plan requires an excessive amount of masonry.

Wall Footing. Assume that a 24-inch wall, with a total load of 42,000 pounds per running foot, is to rest on a soil which can

safely bear a load of 7,000 pounds per square foot. The required width of footing is 6 feet. The footing will project 2 feet on either side of the wall. For each lineal foot of the wall and on each side, there is an inverted cantilever, with an area 2 feet×1 foot, and carrying a load of 14,000 pounds. The center of pressure is 12 inches from the wall; the moment about a section through the face of the wall is 12×14,000, or 168,000 inch-pounds. Using a grade of concrete such that M equals $95bd^2$, p equals. 00675, and j equals .88, then with b equal to 12, we have

$$d^2 = M \div 95b = 168{,}000 \div 1{,}140 = 147.4$$
$$d = 12.15$$

Using this value, the amount of steel required per inch of width will equal .00675×12.15, or .082 square inch, which may be supplied by $\frac{3}{4}$-inch bars spaced about 7 inches on centers. A total thickness of 15 inches will, therefore, fulfil the requirements. Theoretically, this thickness could be reduced to 8 or even 6 inches at the outer edge, since there the moment and the shear both reduce to zero. But when concrete is used very wet and soft, it cannot be laid with an upper surface of even moderate slope without using forms, which would cost more than the saving in concrete.

Shear. The shear (V) on a vertical section directly under the face of the wall, and 12 inches long, is 14,000 pounds. Applying Equation (21)

$$\begin{aligned} v &= V \div bjd \\ &= 14{,}000 \div (12 \times .88 \times 12.15) \\ &= 109 \text{ lb. per sq. in.} \end{aligned}$$

This is far greater than a safe working stress and the slab might fail from diagonal tension. When a loaded beam is supported freely at each end, the maximum shear is found at the ends where the moment is minimum, and some of the bars which are not needed there for moment may be bent up so as to resist the shear. Unfortunately, in the case of a cantilever, the maximum moment and maximum shear are found at the same beam section—in this case, at the face of the wall. Therefore, if the concrete itself cannot carry the shear, additional steel must be used to do that work. Bars which are

This may be done by having them extend through the column and by bending the free ends. Assume that the concrete alone takes up 40 pounds of the 109 pounds shear, found above, or 37 per cent. This leaves 63 per cent to be taken by the steel reinforcement. $14{,}000 \times .63 = 8{,}820$ pounds per foot, or 735 pounds per lineal inch. The only practicable arrangement is to alternate these bars with the moment bars and therefore space them 7 inches apart. Then each bar must take up 7×735, or 5,145 pounds of shear. A $\frac{9}{16}$-inch square bar will safely sustain that stress. Such a bar has a perimeter of 2.25 inches. At 75 pounds per square inch for bond adhesion (plain bars), each lineal inch of the bar would have a working adhesion of 169 pounds. $5{,}145 \div 169 = 30$ inches, which is the required length of bar beyond any point where the stress is as much as 5,145 pounds. Since there is not that length of bar available, bond adhesion cannot be relied on and the bars must be bent, as shown in Fig. 114. Even a deformed bar, although a good type may be used with working adhesion about double that of a plain bar, would need to be longer than space permits, if straight, and it should be hooked.

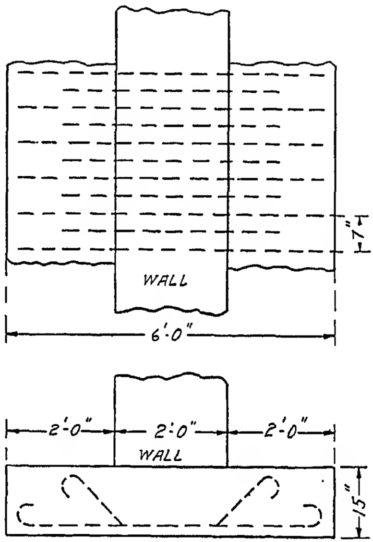


Fig. 114. Diagram of Footing for a Wall

Bond Adhesion in Moment Bars. The steel per inch of width is .082 square inch and in 7 inches, .574 square inch. Since the design calls for a unit tension of 16,000 pounds in the steel, the actual tension in the bar will be $16{,}000 \times .574 = 9{,}184$ pounds. A $\frac{3}{4}$-inch square bar has a perimeter of 3 inches and, at 75 pounds per square inch, can furnish a working bond adhesion of 225 pounds per lineal inch of bar. But this would require $9{,}184 \div 225$, or 41 inches, the required length beyond the face of the wall. Allowing 150 per square inch bond adhesion, for a good type of deformed bar, the required length, computed similarly, would be a little over 20 inches, and as this is less than the 24-inch cantilever, straight deformed

bars will do. The designer, therefore, has the choice of using a hook on each end of plain bars, as illustrated in Fig. 114, or using straight deformed bars, which would be cheaper at the usual relative prices.

Column Footing. The most common method of reinforcing a simple column footing is shown in Fig. 115. Two sets of the reinforcing bars are at *a-a* and *b-b*, and are placed only under the column. To develop the strength of the corners of the footings, bars are placed diagonally across the footing, as at *c-c* and *d-d*. In designing this footing, the projections of the footing beyond the column are treated as free cantilever beams, or by the method discussed above. The maximum shear occurs near the center; and therefore, if it is necessary to take care of this shear by means of reinforcement, it should be provided by using stirrups or bent bars.

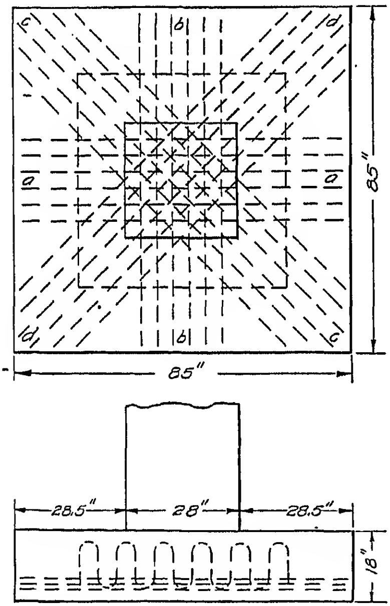


Fig. 115. Diagram of Footing for a Column

Example. Assume that a load of 300,000 pounds is to be carried by a column 28 inches square, on a soil that will safely carry a load of 6,000 pounds per square foot. What should be the dimensions of the footing and the size and spacing of the reinforcing bars? The bars are to be placed diagonally as well as directly across the footing, as illustrated in Fig. 115. Also investigate the shear.

Solution. The load of 300,000 pounds will evidently require an area of 50 square feet. The sides of the square footing will evidently be 7.07 feet, or say 85 inches; and the offset on each side of the 23-inch column is 28.5 inches. The area of each cantilever wing which is straight out from the column is 28.5$\times$28, or 798 square inches. The load is, therefore, (798$\div$144)$\times$6,000, or 33,250 pounds. Its lever arm is one-half of 28.5 inches, or 14.25 inches. The moment is therefore 473,812 inch-pounds. Adopting the straight-line formula, $M=95\,b\,d^2$, on the basis that $p=.00675$, we may write the equation

$$473{,}812=95\times28\times d^2,$$

This area of metal may be furnished by six $\frac{3}{4}$-inch round bars, and therefore there should be six $\frac{3}{4}$-inch round bars spaced about 4.5 inches apart under the column in *both* directions, *a-a* and *b-b*.

Corner Sections. The mechanics of the reinforcements of the corner sections, which are each 28.5 inches square, is exceedingly complicated in its precise theory. The following approximation is probably sufficiently exact. The area of each corner section is the square of 28.5 inches, or 812.25 square inches. At 6,000 pounds per square foot, the pressure on such a section will be 33,844 pounds, and the center of gravity of this section is of course at the center of the square, which is 14.25×1.414, or 20.15 inches from the corner of the column. A bar immediately under this diagonal line would have a lever arm of 20.15 inches. A bar parallel to it would have the same lever arm from the middle of the bar to the point where it passes under the column. Therefore, if we consider that this entire pressure of 33,844 pounds has an average lever arm of 20.15 inches, we have a moment of 681,957 inch-pounds. Using, as before, the moment equation $M = 95bd^2$, we may transpose this equation to read

$$b = \frac{M}{95d^2}$$

Then

$$A = pbd = p\frac{M}{95d^2}d = p\frac{M}{95d}$$
$$= .00675 \times \frac{681{,}957}{95 \times 14.5}$$
$$= 3.34 \text{ sq. in.}$$

This area of steel will be furnished by six $\frac{3}{4}$-inch round bars. The diagonal reinforcement will therefore consist of six $\frac{3}{4}$-inch round bars running diagonally in both directions. These bars should be spaced about 5 inches apart. Those that are nearly under the diagonal lines of the square should be about 9 feet 8 inches long; those parallel to them will each be 10 inches shorter than the next bar.

Bond Adhesion. The total tension in the steel of the *a* and *b* bars is $16{,}000 \times 2.51 = 40{,}160$ pounds, or 6,693 pounds per bar, which is found at a point immediately under the column face. There will be 28.5 inches length of steel in each bar from the column face to the edge of the slab, and this will require a bond adhesion of $6{,}693 \div 28.5 = 235$ pounds per lineal inch. From Table XXIII, we see that this unit value is greater than a proper working value for $\frac{3}{4}$-inch plain round bars but is safe for $\frac{3}{4}$-inch deformed round bars. Making a similar calculation for the diagonal bars, the stress in each one is $(16{,}000 \times 3.34) \div 6 = 8{,}907$ pounds. The length, practically uniform for each, beyond the face of the column is 40 inches, which will require a bond adhesion of 223 pounds per lineal inch. This is just within the limit for $\frac{3}{4}$-inch plain square bars.

It should be noted from the solution of this and the previous problem that, on account of the combination of heavy load and small cantilever projection, the bond adhesion of footings is always a critical matter and its investigation should never be neglected. It frequently happens, as above illustrated, that the greater bond resistance of deformed bars will permit the use of a certain bar which

bars, the requisite adhesion may sometimes be obtained by using a proportionately larger number of smaller bars. When neither method will produce the required adhesion, the bars should be bent into a hook, which should be a full semicircle with a diameter about 8 to 12 times the diameter of the bar.

Shear. The "punching shear" on the slab is measured by the upward pressure on that part of the slab which is outside of the column area. This equals $85^2-28^2=6441$ square inches, or 44.73 square feet. Multiplying by 6,000 we have 268,380 pounds. The resisting area equals the perimeter of the column times jd, which here equals $4\times28\times.88\times13.3$, or 1,311 square inches. Dividing this into 268,380, we have 204 pounds per square inch. If the column and slab were made of plain concrete, this figure would be considered too high for working stress, 120 being usually allowed. In this case, an actual punching of the slab would require that 48 sections of $\frac{3}{4}$-inch round bars should be sheared off. Allowing that the concrete actually takes an average of 120 pounds per square inch on 1,311 square inches of surface, the concrete would take up 157,320 pounds, leaving 111,060 pounds for the 48 bars, or 2,314 pounds for each bar. Dividing by the bar area, we have a shearing stress of 5,237 pounds per square inch of bar section, which is insignificant for the steel and is amply safe, provided that any such shearing stress as 2,314 pounds per bar could be developed before the concrete itself were crushed by the bars. Considering the various forces resisting the punching action, and also that even the 204 pounds per square inch is far short of the ultimate value of *true* shear, the design is probably safe, although the factor of safety is probably low. If further reinforcement were considered necessary, it could be added in the form of bent bars, as in the previous problem.

It is impracticable to develop a true rational formula for the computation of the diagonal tension in slabs which support columns, but the results of a series of elaborate tests by Prof. Talbot (Bulletin No. 67, Univ. of Illinois) show that the following method gives results which are reasonably consistent and also comparable with

the corresponding results for ordinary beams. Consider a section through the slab all the way around the column and at a distance d from the face of the column, and apply Equation (21), $v=\frac{V}{bjd}$. In this case the section would be a square $(2\times13.3)+28=54.6$ inches on a side. The area is 2,981 square inches. The area of the whole footing is 85^2, or 7,225 square inches and the area outside this square is $7{,}225-2{,}981=4{,}244$ square inches, or 29.5 square feet. $29.5\times6{,}000=177{,}000$ pounds $=V$; the perimeter of the square is b and equals 4×54.6, or 218.4; jd equals $.88\times13.3$, or 11.7. Then v equals 69. Since this is higher than 40, the usual permissible working stress when taken as a measure of non-reinforced diagonal tension, it shows that bent bars or stirrups must be used, but in either case the reinforcement need carry only the extra 29 pounds per square inch. Multiplying this by jd, we have $29\times11.7=339$, the required assistance in pounds per lineal inch. If a bar is placed every 4.5 inches (corresponding with the main reinforcing bars), the stress per bar will be 1,525 pounds, which at 16,000 pounds unit stress will require .095 square inches, or a $\frac{5}{16}$-inch square bar. Perhaps the most convenient form of reinforcement in this case would be a series of stirrups made by a continuous bar $\frac{5}{16}$ inch square, which zigzags up and down with an amplitude equal to jd, or 11.7 inches, and so that there is a bar up or down each 4.5 inches. This should be located at the "critical section" at a distance d equal to 13.3 inches from the column face. It will require a bar about 16 feet 6 inches long to make the continuous stirrup for each side of the square. Each bar must be bent with about eleven semicircular bends, as shown in Fig. 115, and so placed that each downward loop shall pass under one of the main reinforcing bars. The loops at the top will preclude all possibility of bond failure.

Since the shear decreases to zero at the edge of the slab, and the distance from the stirrup to the edge of the slab is only a little more than the thickness of the slab, it is apparent without calculation that no further shear reinforcement is needed.

Continuous Beams. Continuous beams are sometimes used to save the expense of underpinning an adjacent foundation or wall. These footings are designed as simple beams, but the steel is placed

columns are 22 inches square, spaced 12 feet on center; and that they support a load of 195,000 pounds each. If the soil will safely support 6,000 pounds per square foot, the area required for a footing will be 195,000÷6,000, or 32.5 square feet. Since the columns are spaced 12 feet apart, the width of footing will be 32.5÷12=2.71 feet, or 2 feet 9 inches. To find the depth and amount of reinforcement necessary for this footing, it is designed as a simple inverted beam supported at both ends (the columns), and loaded with an upward pressure of 6,000 pounds per square foot on a beam 2 feet 9 inches wide. In computing the moment of this beam, the continuous-beam principle may be utilized on all except the end spans, and thus reduce the moment and, therefore, the required dimensions of the beam.

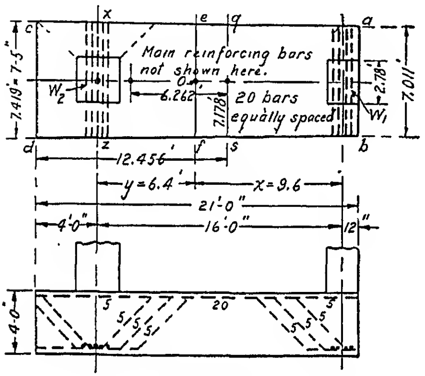


Fig. 116. Combined Footing for Two Columns, One on Edge of Property

Compound Footing. When a simple footing supports a single column, the center of pressure of the column must pass vertically through the center of gravity of the footing, or there will be dangerous transverse stresses in the column, as discussed later. But it is sometimes necessary to support a column on the edge of a property when it is not permissible to extend the foundations beyond the property line. In such a case, a simple footing is impracticable. The method of such a solution is indicated in Fig. 116. The nearest interior column (or even a column on the opposite side of the building, if the building be not too wide) is selected, and a combined footing is constructed under both columns. The weight on both columns is computed. If the weights are equal, the center of gravity is halfway between them; if unequal, the center of gravity is on the line joining their centers, and at a distance from them such that $x:y::W_2:W_1$, Fig. 116. In this case, evidently W_2 is the greater weight. The area $abdc$ must fulfill two conditions:

(1) The area must equal the total loading (W_1+W_2) divided by the allowable loading per square foot; and,

(2) The center of gravity must be located at O.

An analytical solution for all cases of the relative and absolute values of ab and cd which will fulfill the two conditions is very difficult. Sometimes the only practicable solution is to obtain, by trial and adjustment, a set of dimensions which will be sufficiently accurate for practical purposes. It usually happens that an inner column of a building carries a greater load than an outer column. This facilitates the solution, for then, as in the example given below, the footing may be extended beyond the inner column and may be made approximately rectangular.

Example. A column W_1, carrying 400,000 pounds, is to be located on the edge of a property and another column W_2, carrying 600,000 pounds, is located 16 feet from it. Assume that the subsoil can sustain safely 7,000 pounds per square foot. Required the shape and design of the footing.

Solution. Assume that the footing slab weighs 400 pounds per square foot of surface; then the *net* effective upward pressure of the subsoil which will support the column equals $7{,}000-400=6{,}600$ pounds per square foot. For simplicity of calculation in the computations involving soil pressures and slab areas, feet and decimals will generally be used. The change to feet and inches can be made when the final dimensions have been computed.

The total column load is 1,000,000 pounds; at 6,600 pounds per square foot the area must be 151.515 square feet. Assume that the W_2 column is 2.89 feet square, and that the W_1 column is 2 feet$\times$2.78 feet. This means that the net average load is 500 pounds per square inch on each column. In Fig. 116, let ab equal n, and cd equal m, both still unknown. The smaller column is on the edge of the property, and the ab line is made 1.0 foot from the column center. As a trial solution, assume that the cd line is 4.0 feet beyond the other column center. Then the total length of the trapezoid is 21.0; then $\frac{1}{2}(m+n)\,21.0=151.515$; solving this

$$(m+n)=14.43$$

The center of gravity of the two loads is at $\dfrac{600{,}000}{1{,}000{,}000}$ of 16 feet, or at 9.6 feet from the smaller column center. This locates O. To fulfill condition (2), the dimensions m and n must be such that the center of gravity of the trapezoid shall be at O. In general, the distance z of the center of gravity of a trapezoid from its larger base equals one-third of the height h times the quotient of the larger base, plus twice the smaller base divided by the sum of the bases; or, as an equation

$$z=\tfrac{1}{3}h\frac{m+2n}{m+n}$$

Substituting z equals 10.4, h equals 21.0, m and n still unknown, we have

Combining this equation with the equation $(m+n)=14.43$, we may solve and find $m=7.419$ and $n=7.011$. By proportion, we find the dimension ef through $O=7.217$ feet.

Moment. The maximum moment is found where the shear is zero, and this must be at the right-hand end of a portion of the slab on which the *net* upward pressure equals 600,000 pounds. That portion must have an area of $(600{,}000 \div 6{,}600)=90.909$ square feet. Similarly, the remaining area is computed to be 60.606 square feet. Let p equal the length of this section (qs in the figure) and h equal its distance from cd. We may write the two equations

$$\tfrac{1}{2}(7.419+p)\,h=90.909$$

and

$$\tfrac{1}{2}(p+7.011)(21-h)=60.606$$

Solving these two equations for p and h, we have $p=7.178$ and $h=12.456$. It should be noted that this section of maximum moment (on the line qs) is *not* on the line of center of gravity of the whole footing, but is in this case about two feet to the right. The center of gravity of the trapezoid $cdqs$, calculated as above, is at a point 6.262 feet from qs and the *net* upward pressure on this section is 600,000 pounds. Therefore, taking moments about qs, we have

$$M=600{,}000\ (8.456-6.262)=1{,}316{,}400 \text{ ft.-lb.}=15{,}796{,}800 \text{ in.-lb.}$$

In this case, $b=7.178$ feet $=86.136$ inches; call it even 86. Then for $M=95\,b\,d^2$, we have

$$95\,b\,d^2=8170\,d^2=15{,}796{,}800;\ \text{then } d^2=1{,}934 \text{ and } d=44.0$$

Then

$$A=.00675\times 86\times 44=25.54 \text{ sq. in.}$$

which may be provided by 20 bars, $1\frac{1}{8}$ inches square.

That portion of the slab between x and z is subject to transverse stress, the parts near x and z tending to bend upward. Although the stresses are not computable with perfect definiteness, being comparable to those in a simple footing (see page 249), we may consider them as approximately measured by the moment of the quadrilateral between the face of the column and x about the face of the column. xz equals 7.34; subtracting the column width and dividing by 2, we have 2.225 feet, or 26.7 inches; the area of the quadrilateral is approximately $\frac{1}{2}(8+2.89)2.225$, or 12.11 square feet. The effective upward pressure equals $12.11\times 6{,}600=79{,}926$ pounds. The lever arm is approximately $\frac{6}{10}$ of the distance from the face, or $0.6\times 26.7=16$ inches. $M=79{,}926\times 16=1{,}278{,}816=95\,b\,d^2$. Here d is about one inch less than for the main slab, or say 43 inches. Solving, $b=7.3$ and $A=p\,b\,d=.00675\times 43\times 7.3$, or 2.12 square inches, which may be supplied by 4 bars $\frac{3}{4}$-inch square. This calculation shows that a relatively small amount of reinforcement, which should run under the column from x to z, will resist this stress. Increasing the number of bars to 5 or 6 will certainly cover all uncertainties in this part of the calculation. The stresses under the other column are somewhat less and therefore the same reinforcement will be even more safe.

Shear. The shear around the larger column can be calculated as "punching" shear. b for this case is the perimeter of the column, and equals $4\times 2.89=11.56$ feet, or 138.72 inches; jd equals $.88\times 44=38.72$; V equals 600,000 −

$(138.72 \times 38.72) = 102$. Since this is a case of true shear, when a working stress of 120 pounds per square inch is allowable, no added reinforcement is necessary. The other column may be considered similarly, except that it is supported only on three sides. $b = 81$ inches, and $bjd = 3{,}136$; $V = 300{,}000 - 36{,}667 = 263{,}333$; then v equals 84. Since this is only 70 per cent of the allowable stress for true shear, it is probably safe. In addition, the bending down of the main reinforcing bars under each column, as shown in the figure, will add a very large factor of safety.

Case Where Heavier Column Is Next to the Property Line. It is far more difficult, in case the heavier column is next to the property line, to obtain, by the analytical method given above, a trapezoid which will fulfill the two fundamental requirements there given. If the wall column has twice (or more than twice) the load carried by the inner column, no trapezoid is obtainable. In such a case, a figure shaped somewhat like a shovel, the blade being under the heavy column and the handle being a beam which transfers the load of the lighter column to the broad base, may be used, the dimensions and exact shape of which can only be determined by successive trials.

REINFORCED CONCRETE RETAINING WALLS

Forms of Walls. Reinforced concrete walls are usually made in such shape that advantage is taken of the weight of part of the material supported to increase the stability of the wall against overturning. Fig. 117 shows the outline of such a wall. It consists of a vertical wall CD, attached to a floor plate AB. To prevent the wall from overturning, the moment of downward forces about the outer edge of the base $M = W_1 l_1 + W_2 l_2$ must be greater than that of the overturning moment $M_2 = E l_3$. M_1 should be from one and one-half to twice M_2, which would be the factor of safety. In addition to this factor of safety there would be the shearing of the earth along the line ab.

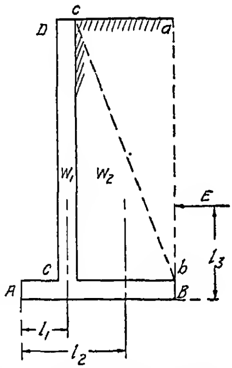


Fig. 117. Outline of Reinforced Concrete Wall

Owing to the skeleton form of these walls they are usually more economical to construct than solid walls of masonry. The cost per cubic yard of reinforced concrete in the wall will be more than the cost per cubic yard of plain concrete or stone, in a gravity retaining wall, but the quantity of material required will be reduced by 30 to 50 per cent in most cases. There are two forms of these walls.

ples of both types will be worked out in detail.

Illustrative Example. Design a retaining wall 14 feet high to support an earth face with a surcharge at a slope of $1\frac{1}{2}$ to 1.

The width of the base for reinforced concrete walls is usually made from $\frac{4}{10}$ to $\frac{6}{10}$ of the height. For this wall, with a surcharge, the base will be made one-half of the height, or $14\times\frac{1}{2}=7$ feet.

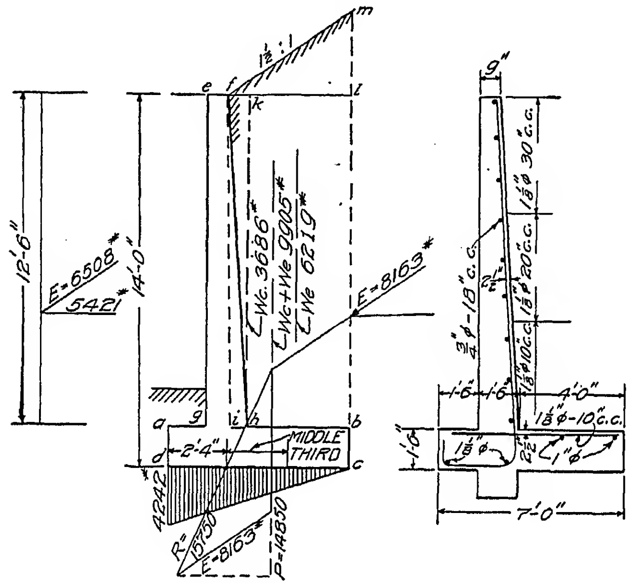


Fig. 118. Design Diagrams for Retaining Wall

Assume the weight of the earth at 100 pounds per cubic foot and the reinforced concrete at 150 pounds per cubic foot. Then substituting in Equation (7c), we have

$$E=.833\frac{Wh^2}{2}$$

$$=.833\times\frac{100\times14^2}{2}$$

$$=8{,}163 \text{ lb.}$$

This force is applied on the plane cm, Fig. 118, and at a point one-third of the height above the base.

It will be necessary to determine the thickness of the vertical wall and the base plate before the stability of the wall can be determined. Assume the base plate to be 18 inches thick; then the vertical slab will be 12 feet 6 inches high and the pressure against this slab will be

$$E = .833\frac{(100\times\overline{12.5}^2)}{2} = 6{,}508 \text{ lb.}$$

The horizontal component of this pressure is 6,508 $\times$ cos 33° 42′, or 5,421 pounds, as shown diagrammatically in Fig. 118. The bending moment will be $M = 5{,}421\times\frac{12.5}{3}\times 12 = 271{,}272$ inch-pounds. Placing this equal to $M = 95\ bd^2$ (see page 192) with b equals 12, d^2 equals 238, and d equals 15.4 inches. Adding 2.6 inches for protecting this steel, the total thickness will be 18 inches. The area of the reinforcing steel will be .00675 $\times$ 15.4, or .104 square inch of steel per inch of length of wall. Bars $1\frac{1}{8}$ inches round (.99 $\div$ .10 = 9.9) spaced 10 inches apart, will be required. The bending moment rapidly decreases from the bottom of the slab upwards, and, therefore, it will not be necessary to keep the thickness of 18 inches to the top of the slab or to have all the bars the full length. Make the top 9 inches thick, drop off one-third of the bars at one-third of the height of the slab and one-third at two-thirds of the height. The shear at the bottom of the slab is $\frac{5{,}421}{12\times 15.4} = 29$ pounds per square inch; therefore, as this does not exceed the working stress, no stirrups are needed. It is very important in a wall of this type not to exceed the bonding stress.

The vertical bars must be well anchored in the base plate or they will be of no great value. The bars are $1\frac{1}{8}$ inches in diameter, the circumference then is 3.53 inches. Allowing a bonding stress of 75 pounds per square inch, the total bonding per inch of length of bar is 3.53 $\times$ 75, or 265 pounds. The lever arm is 15.4. Since the bars are spaced 10 inches on centers, the stress to be resisted is $\frac{5}{6}$ of 271,272, or 226,060 inch-pounds. Let x be length of anchorage required, then

$$M = 265\times 15.4\times x = 226{,}060$$
$$x = 55 \text{ inches}$$

55 inches or be anchored in such a way that their strength will be developed.

In designing the footing of a reinforced concrete retaining wall the resultant force should intersect the base within the middle third the same as in a masonry wall. The forces acting on the footing are the earth pressure on the plane *mc*, the weight of the earth fill, and the weight of the concrete. The distance from the toe *a* to the point where the resultant acts is obtained as follows: The centers of gravity of the concrete and the earth are found, also the weight of each. The weights are multiplied by the distances from *a*, respectively, which gives the static moment. The sum of the static moments divided by the sum of the weights equals the distance from the toe to the line at which the resultant acts. The detail figures for the problem are given below.

Center of Gravity of Wall

Section	Area Sq. Ft.	Moment Arm	Moment Area
a b c d	10.50	3.50	36.75
e f i g	9.38	1.88	17.63
f i h	4.69	2.50	11.73
	24.57		66.11

Distance from a to center of gravity is $\frac{66.11}{24.57} = 2.69$ ft.

Weight per lineal foot is $24.57 \times 150 = 3{,}686 = W_c$

Static moment about a is $3{,}686 \times 2.69 = 9{,}915$ ft.-lb.

Center of Gravity of Earth

Section	Area Sq. Ft.	Moment Arm	Moment Area
f k h'	4.69	2.75	12.90
h b l k	50.00	5.00	250.00
f l m	7.50	5.42	40.65
	62.19		303.55

Distance from a to center of gravity is $\frac{303.55}{62.19} = 4.88$ ft.

Weight per lineal foot is $62.19 \times 100 = 6{,}219 = W_e$

$$\frac{9{,}915+30{,}355}{3{,}686+6{,}219}=\frac{40{,}270}{9{,}905}=4.06 \text{ ft.}$$

To find where the resultant R cuts the base, produce E to meet the combined center of gravity of the concrete and earth. From their intersection lay off on the vertical line, at any convenient scale, the combined weight 9,905 pounds. At the end of this distance draw a line parallel to the line E and lay off the value of E which is 8,163 pounds. Draw R, which is the resultant and in this case cuts the base at the edge of the middle third, so that the wall will not fall by overturning.

The pressure produced on the foundation is next to be investigated. Since the resultant comes at the edge of the middle third, Equations (**7d**) and (**7e**) are used.

$$\text{Pressure at the toe} = (4B-6Q)\,\frac{P}{B^2}$$

$$=[(4\times 7)-(6\times 2.33)]\,\frac{14{,}850}{7^2}$$

$$=4{,}242 \text{ pounds}$$

$$\text{Pressure at the heel} = (6Q-2B)\,\frac{P}{B^2}$$

$$=[(6\times 2.33)-(2\times 7)]\,\frac{14{,}850}{7^2}$$

$$=0$$

The pressure on the foundation of 4,242 pounds at the toe is permissible on most soils.

The stability of a wall of this type must be carefully investigated. Suppose this wall is to be located on a wet clay soil. The coefficient of friction between concrete and wet clay is .33; the horizontal force is 6,800 pounds; and the weight of the concrete and earth acting in a downward direction is 9,915 pounds. With a coefficient of .33, or $\frac{1}{3}$, the resistance to sliding is $9{,}915\times\frac{1}{3}$, or 3,305 pounds, which is less than one-half of the horizontal pressure 6,800. The resistance should be about twice the pressure in order to make the wall safe against sliding, which would require that the weight

should be about four times as much in order that mere friction should surely prevent sliding. This shows that it will be necessary to construct a projection in the base, as shown in Fig. 118.

The thickness of the base is always made greater than the moment requirements just behind the vertical slab (or at h) would demand. If the wall were actually on the point of tipping over, there would cease to be any upward pressure on the base. But there would be a downward pressure on the right cantilever equal to the weight of the earth above it, and the moment in the base at the point h would be that produced by that earth pressure and by the weight of the concrete from h to b. Since the above calculations for the stability of the wall show that the computed lateral pressure cannot produce actual tipping about the toe, no such moment can actually be developed, but the calculation of the required thickness to resist such a moment gives a dimension which is certainly more than safe and which, for other reasons, is sometimes made still greater. The weight of the earth is 6,229 pounds and the weight of the concrete is $4 \times 1\frac{1}{2} \times 150 = 900$ pounds. Then $6{,}229 + 900 = 7{,}129$ pounds. Therefore

$$M = 7{,}129 \times 1.86 \times 12 = 158{,}977 \text{ in.-lb.}$$

Placing this moment equal to $M = 95\, bd^2$ and solving for d, we find that d equals 11.7. Adding 2.5 inches for protecting the steel, the total thickness would be 14.2 inches. To properly anchor the bars in the vertical slab, the thickness of base plate is seldom made less than the vertical slab. Therefore, we will make $d = 15$ inches, $b = 12$, and solve for the moment factor R.

$$M = 12 \times \overline{15}^2 \times R = 158{,}977$$
$$R = 58.8$$

Fig. 99 shows that when $R = 59$, $C = 400$ and $S = 12{,}000$ and that the percentage of steel required is practically .006. Therefore, the steel required equals $12 \times 15 \times .006 = 1.08$ square inches. Bars $1\frac{1}{8}$ inches in diameter, spaced 10 inches, will be required. The moment in this part of the base plate is negative, therefore the steel must be placed in the top of the concrete.

The vertical shear is $\dfrac{7{,}129}{12 \times 15}$, or 39 pounds per square inch, which

The left cantilever or toe has an upward pressure. At the extreme end it is 4,240 pounds and at the face of the vertical wall it is 3,200—scaled from Fig. 118. The average pressure is (4,240+3,200)÷2=3,720 pounds. The moment is, therefore,

$$M = 3{,}720 \times \frac{1.5}{2} \times 12 = 33{,}480 \text{ in.-lb.}$$

Let $d=15$, $b=12$, and solve for R

$$12 \times \overline{15}^2 \times R = 33{,}480$$
$$R = 12.4$$

This value of 12.4 for R is smaller than is found in Fig. 99. Since the bars in the vertical slab are bent in such a shape as to supply this tension, no further consideration of this stress is necessary in this problem.

Some longitudinal bars must be placed in the wall to prevent temperature cracks, and also to tie the concrete together. About .003 per cent of the area above the ground is often used. In this case $\frac{3}{4}$-inch round bars spaced 18 inches on centers will be used.

Reinforced Concrete Retaining Walls with Counterforts. In this type of wall the vertical slab is supported by the counterforts, the principal steel being horizontal. The counterforts act as cantilever beams, being supported by the footing.

Illustrative Example. Design a reinforced-concrete wall with counterforts, the wall to be 20 feet high and the fill to be level with the top of the wall.

The spacing of the counterforts is first determined. The economical spacing will vary from 8 feet to 12 feet or more, depending on the height of the wall. A spacing of 9 feet on centers will be used for the counterforts in this case, Fig. 119. The maximum load on the slab is on the bottom unit and decreases uniformly to zero at the top, when the earth is horizontal with the top of the wall, as in this case. Assume that the base plate will be 18 inches in thickness, then the center of the bottom foot of slab will be 18 feet from the top of the wall. Then pressure to be sustained by the lower foot of the slab will be

in which P is the intensity of the horizontal pressure at any depth h, and w is the weight per cubic foot of the earth.

$$P=\frac{1}{3}\times100\times18$$

$$=600 \text{ pounds per square foot}$$

Multiplying this value of P by the distance between the centers of the counterforts—$600\times9=5{,}400$—the full load is obtained.

$$M=\frac{5{,}400\times9\times12}{8}=72{,}900 \text{ in.-lb.}$$

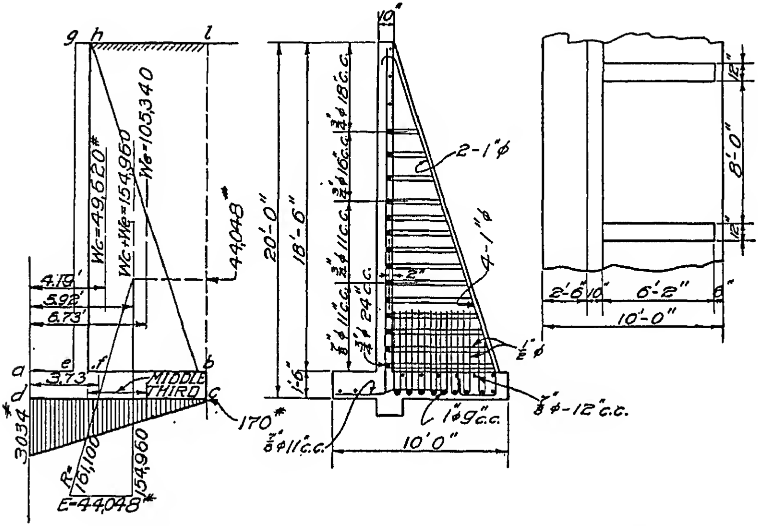


Fig. 119. Design Diagrams for Retaining Wall with Counterforts

Placing this value of M equal to $95bd^2$ in which $b=12$, and solving for d, we have

$$95\times12\,d^2=72{,}900$$

$$d^2=64$$

$$d=8$$

Adding 2 inches to this—$8+2=10$—for protecting the steel, the total thickness of the wall will be 10 inches. For convenience of construction the slab will be made uniform in thickness. The steel for the bottom inch will be $.00675\times18=.054$ square inch. $.60\div.054=11$ inches. That is, $\frac{7}{8}$-inch round bars may be spaced 11 inches on centers. Use this size of bars and spacing for

five per cent, and $\frac{3}{4}$-inch round bars, spaced 11 inches, will be used. In the third quarter, the required area will be one-half of that required for the first quarter. $.054 \div 2 = .027$ square inch, or $.44 \div .027 = 16$, that is, $\frac{3}{4}$-inch round bars spaced 16 inches on centers should be used. In the upper part of the wall use $\frac{3}{4}$-inch round bars, 18 inches on centers.

To determine the requirements of the counterforts it will be necessary to determine the horizontal pressure against a section of the wall nine feet long. Referring to page 153, Part II, we see that Equation (7) is stated thus:

$$E = \tan^2 \left(45^\circ - \frac{\phi}{2}\right) \frac{W h^2}{2}$$

Substituting in the modified form of Equation (7a) and multiplying by 9

$$E = .286 \times \frac{100 \times (18\tfrac{1}{2})^2}{2} \times 9$$

$$= 44{,}048 \text{ lb.}$$

This load is applied at one-third of the height of the wall, which is 6.5 feet above the base. The moment in the counterfort is

$$M = 44{,}048 \times 6\tfrac{1}{2} \times 12$$
$$= 3{,}435{,}744 \text{ in.-lb.}$$

The width of counterfort must be sufficient to insure rigidity, to resist any unequal pressures, and to thoroughly embed the reinforcing steel. The width is made by judgment and in this case will be made 12 inches wide. The counterfort and vertical slab together form a T-beam with a depth at the bottom of 84 inches. Allow 4 inches to the center of the steel, then $d = 80$ inches; $jd = .87d = .87 \times 80 = 69.6$ inches.

$$M = A_s \times jd \times 16{,}000$$
$$3{,}435{,}744 = A_s \times 69.6 \times 16{,}000$$
$$A_s = 3.0 \text{ sq. in.}$$

Four one-inch round bars will give this area. Two of these bars will extend to the top of the wall and two may be dropped off at half the height.

Now that these dimensions have been determined, the wall will

be investigated for stability against overturning. Substituting in Equation (7a)

$$E = .286\,\frac{W h^2}{2}$$

$$= .286 \times \frac{100 \times \overline{20}^2}{2}$$

$$= 5{,}720$$

To find the center of gravity of the wall, it will be necessary to take a section 9 feet long, that is, center to center of counterforts.

Center of Gravity of Concrete

Moments taken about A

Section	Volume Cu. Ft.	Moment Arm	Volume Moment
$a\ b\ c\ d$	135.0	5.0	675.0
$e\ f\ h\ g$	138.8	2.92	405.3
$h\ f\ b$	57.0	5.538	306.7
	330.8		1,387.0

Distance from a to center of gravity $\frac{1{,}387.0}{330.8} = 4.19$ ft.

Weight of 9 feet of wall $= 330.8 \times 150 = 49{,}620$ lb.

Static moment about a for section 9 feet long, $49{,}620 \times 4.19 = 207{,}908$ ft.-lb.

Center of Gravity of Earth

Moments about A

Section	Volume Cu. Ft.	Moment Arm	Volume Moment
$f\ b\ l\ h$	987.0	6.66	6,573.4
$b\ l\ h$	66.4	7.77	515.9
	1,053.4		7,089.3

Distance from a to center of gravity $\frac{7{,}089.3}{1{,}053.4} = 6.73$ ft.

Weight of earth per 9 feet of wall $1{,}053.4 \times 100 = 105{,}340$ lb.

Static moment about a, for section 9 feet long equals $105{,}340 \times 6.73 = 708{,}930$ ft.-lb.

Distance from a to the resultant of the concrete and earth

$$\frac{207{,}908 + 708{,}930}{49{,}620 + 105{,}340} = \frac{916{,}838}{154{,}960} = 5.92 \text{ ft.}$$

Draw the line W_c+W_e at a distance 5.92 feet from A and produce the line E to meet it. From the intersection of these two lines lay off the sum of the weight of the concrete plus the weight of the earth at any convenient scale. At the end of this distance draw a line parallel to E and lay off on it the value found for E. Draw the resultant R. This line produced on to the base falls within the middle third, and therefore, the wall should be safe against overturning.

Since the resultant cuts the base within the middle third, Q is greater than one-third of the width of the base and Equations **(7d)** and **(7e)** will be applied in finding the pressure on the base. Substituting in Equation **(7d)**

$$\text{Pressure at the toe} = (4B-6Q)\frac{P}{B^2}$$

$$=(4\times 10-6\times 3.73)\frac{154{,}960}{10^2}$$

$$=27{,}304 \text{ lb.}$$

Dividing 27,304 by 9 we have 3,034 pounds, which is the weight per foot in length of the wall on the toe.

The pressure at the heel is found by substituting in Equation **(7e)**

$$\text{Pressure at the heel} = (6Q-2B)\frac{P}{B^2}$$

$$=(6\times 3.73)-(2\times 10)\frac{154{,}960}{10^2}$$

$$=3{,}688 \text{ lb.}$$

Dividing 3,688 by 9 gives 410 pounds, which is the weight per lineal foot at the heel.

In designing the toe (left cantilever) there is the average pressure, $(3{,}034+2{,}378)\div 2=2{,}706$, for which steel must be provided.

$$2{,}706\times 2.5=6{,}765$$

$$M=6{,}765\times\frac{2.5}{2}\times 12=101{,}475$$

With $b=12$ and $d=15$ (the total thickness allowed was 18 inches), and solving for R, we have

Therefore $C=300$ and $S=12{,}000$, approximately, and $p=.0035$.

$12\times15\times.0035=.63$ square inches of steel per lineal foot of wall, which is equal to $\frac{7}{8}$-inch round bars spaced 11 inches on centers. As a precaution against the load being concentrated under the counterforts, three extra bars should be placed in the toe at these places.

The rear portion of the footing is designed as a simple beam between the counterforts. It must have sufficient strength to support the earth above it and also its own weight, although, as explained previously for the L-shaped wall, such a stress cannot be developed unless the wall were just at the point of overturning, and the investigation for stability shows that this cannot happen. The following calculation therefore introduces an additional factor of safety in the design of the base slab of perhaps 2, in addition to the usual working factor of about 4.

$$\begin{aligned} \text{Weight of earth} &= 105{,}340 \\ \text{Weight of base} &= \underline{\ \ 13{,}500} \\ & \quad\ 118{,}840 \text{ lb.} \end{aligned}$$

$$M=\frac{118{,}840\times9\times12}{8}=1{,}604{,}340$$

With $b=80$ and $d=15$, solve for R

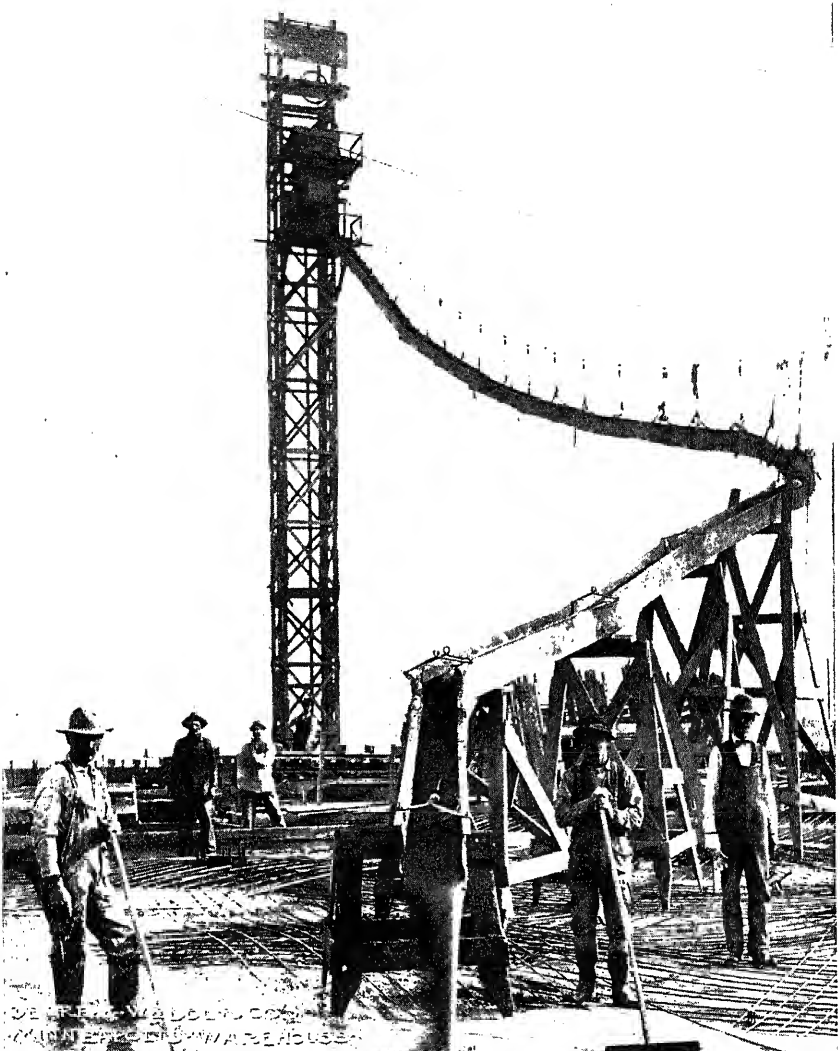
$$80\times\overline{15}^2\times R=1{,}604{,}340$$
$$R=89$$

From Fig. 99 we find, with steel stressed to 16,000 pounds, the concrete would be stressed to about 575 pounds per square inch and the required percentage of steel of .0062 will be required.

$$.0062\times80\times15=7.44 \text{ sq. in.}$$

Nine bars 1 inch round, spaced 8 inches apart, will be required.

In addition to the steel that has been required to satisfy the different equations, the bars in the vertical slab and those in the rear portion of the footing must be tied to the counterforts. (See Fig. 119.) A few bars should also be placed in the top of the footing, but no definite calculation can be made for them. The vertical slab should be reinforced for temperature stresses. In this wall



SPREADING CONCRETE OVER REINFORCING STEEL BY MEANS OF TOWER AND DISTRIBUTING CHUTE

Courtesy of Leonard Construction Company, General Contractors, Chicago

Coping and Anchorages. Retaining walls generally have a coping at the top. This can be made to suit the conditions or the designer. When reinforced concrete walls are not stable against sliding, they can be anchored by making a projection of the bottom into the foundation. This is shown in Figs. 118 and 119.

VERTICAL WALLS

Curtain Walls. Vertical walls which are not intended to carry any weight are sometimes made of reinforced concrete. They are then called curtain walls, and are designed merely to fill in the panels between the posts and girders which form the skeleton frame of the building. When these walls are interior walls, there is no definite stress which can be assigned to them, except by making assumptions that may be more or less unwarranted. When such walls are used for exterior walls of buildings, they must be designed to withstand wind pressure. This wind pressure will usually be exerted as a pressure from the outside, tending to force the wall inward; but if the wind is in the contrary direction, it may cause a lower atmospheric pressure on the outside, while the higher pressure of the air within the building will tend to force the wall outward. It is improbable, however, that such a pressure would ever be as great as that tending to force the wall inward. Such walls may be designed as slabs carrying a uniformly distributed load and supported on all four sides. If the panels are approximately square, they should have bars in both directions and should be designed by the same method as "slabs reinforced in both directions", as has previously been explained. If the vertical posts are much closer together than the height of the floor, as sometimes occurs, the principal reinforcing bars should be horizontal, and the walls should be designed as slabs having a span equal to the distance between the posts. Some small bars spaced about 2 feet apart should be placed vertically to prevent shrinkage. The pressure of the wind, corresponding to the loading of the slab, is usually considered to be 30 pounds per square foot, although the actual wind pressure will very largely depend on local conditions, such as the protection which the building receives from surrounding buildings. A pressure of thirty

desirable to make it any thinner. Since designing such walls is such an obvious application of the equations and problems already solved in detail, no numerical illustration will here be given.

CULVERTS

A flat slab design is generally used for spans up to 20 feet for both highway and railroad culverts. In highway construction, it is some-

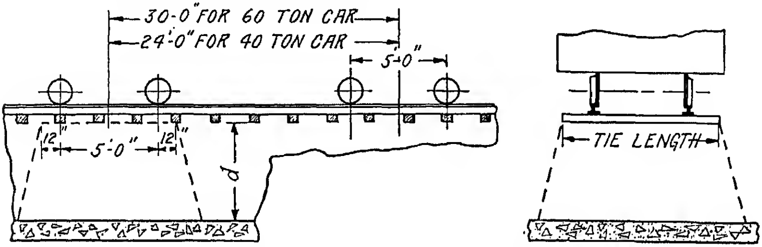


Fig. 120. Load Diagram for 60-Ton and 40-Ton Electric Cars

times found more economical to use the girder bridge for spans as short as 14 or 16 feet. This discussion will be confined to box culverts for highway use. Concrete, and particularly reinforced concrete, is now much used for culverts and bridges. Its permanence and freedom from maintenance charges, compared with wood and with steel structures, is much in its favor.

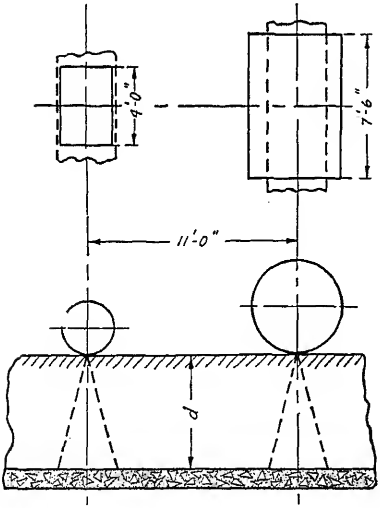


Fig. 121. Load Diagram for Road Roller

Classification by Loadings. Highway structures are usually divided into three classes, as follows:

Class No. 1. Light highway structures for ordinary country use where the heaviest load may be taken as a 12-ton road roller. The uniform live load 100 pounds per square foot.

Class No. 2. Heavy highway structures for use where 20-ton road rollers and electric cars of a minimum weight of 40 tons must be

form distributed load 150 pounds per square foot.

Load Diagrams. Diagrams representing the loadings for 40- and 60-ton cars and for road rollers are shown in Figs. 120 and 121, respectively. Since short-span structures are being considered, only one truck of a car will be on the culvert at one time. The truck of a car will be considered as distributing the load over an area 2 feet longer than the center to center of the wheels, and of a width equal to the length of the ties, which is usually 8 feet. The fill will further distribute this load on a slope of $\frac{1}{4}$ to 1. The fill over a culvert should never be less than 1 foot. For fast-moving cars the bending moment for the live load should be increased 35 per cent for impact when the fill is less than 5 feet.

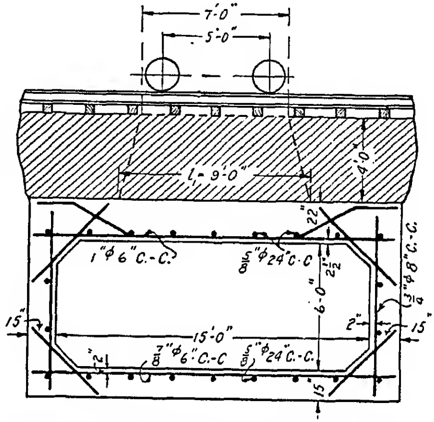


Fig. 122. Design Diagram for Flat-Slab Culvert with 15-Foot Span

Example. Design a flat-slab culvert with a span of 15 feet to support a fill of 4 feet under the ties, a macadam roadway, and a 40-ton car.

Solution. The top will be considered first and a width of 1 foot will be taken. The fill at 100 pounds per cubic foot will equal $100\times4\times15=6{,}000$ pounds. The macadam would have a thickness of the rail plus the tie, which will be about 12 inches. This material at 125 pounds per cubic foot would equal $125\times1\times15=1{,}875$ pounds for a strip 1 foot wide. The maximum bending moment for the live load will occur when one of the trucks of a car is at the middle of the span. The load, 20 tons, will be distributed over an area, as shown in Fig. 122, 9 feet by 10 feet $=90$ square feet. A strip 1 foot wide then must support $20\times2{,}000\div10=4{,}000$ pounds. The formula for this bending moment would be

$$M=\left(\frac{We}{4}-\frac{We_1}{8}\right)12$$

Substituting in this formula, we have

$$M_l=\left(\frac{4{,}000\times15}{4}-\frac{4{,}000\times9}{8}\right)12=126{,}000$$

Add 30 per cent for impact	37,800
Total moment for live load	163,800

Assume that the slab will be 22 inches thick, then a strip 1 foot wide weighs $1\frac{5}{6}\times15\times150=4{,}125$ pounds. The total weight of the fill, macadam and concrete, is 12,000 pounds. The moment for this load is

$$M=\frac{12{,}000\times15\times12}{8}=270{,}000 \text{ in.-lb.}$$

Moment for live load	163,800 in.-lb.
Total moment	433,800

Placing this moment equal to $95\,bd^2$, where $b=12$, we have

$$433{,}800=95\times12\times d^2$$
$$d^2=380$$
$$d=19.5 \text{ inches}$$

Add $2\frac{1}{2}$ inches for protecting the steel, then the total thickness will be 22 inches. The steel required equals $.00675\times12\times19.5=1.58$ square inches. Round bars 1 inch in diameter, spaced 6 inches on centers, will satisfy this requirement.

The shear at the point of supports will equal one-half the sum of the live and dead loads divided by the area of the section.

$$\frac{12{,}000+4{,}000}{2}=8{,}000$$

$$v=\frac{8{,}000}{bjd}=\frac{8{,}000}{12\times.87\times19.5}=39 \text{ lb. per sq. in.}$$

which is much less than the permissible working load. Even in this case one-third of the bars should be turned up at about 3 feet from the end of the span.

The horizontal pressure on the side walls of the culvert produced by the earth will vary with the depth below the surface. The center of the top foot of the side walls is 7.5 feet and the center of the bottom foot is 12.5 feet below the surface of the roadway. Substituting in Equation (7)

At the top $$P=\frac{Wh}{3}=\frac{100\times7\frac{1}{2}}{3}=250 \text{ lb. per sq. ft.}$$

At the bottom $$P=\frac{100\times12\frac{1}{2}}{3}=416 \text{ lb. per sq. ft.}$$

The average pressure equals $(250+416)\div2=333$ pounds. This is not strictly accurate but sufficiently so for the side walls. The live load is $4{,}000\div9=444$ per square foot. It will be assumed that the horizontal pressure from the live load equals $444\div3=148$ pounds per square foot, this load being independent of the depth of the fill. The total live and dead load is, therefore, $333+148=481$ pounds per square foot.

$$M=\frac{481\times6^2\times12}{8}=25{,}974 \text{ in.-lb.}$$

A slab with a thickness of 7 inches would satisfy this equation. Since the side walls must support the top slab as well as the side pressures, they should not be much less in thickness than the top. Make the walls 15 inches thick and reinforce them as shown in Fig. 122.

The bottom is sometimes made the same as the top. This is not necessary unless the foundation is very soft and the load must be distributed over the whole area. In this case it will be made the same as the side walls and reinforced as shown.

The fillets in the corners will assist in stiffening the structure. Wing walls must be provided at the ends. Longitudinal reinforcement also must be provided.

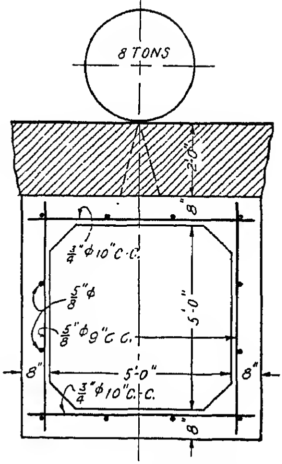


Fig. 123. Design Diagram for Box Culvert 5 Feet Square

Example. Design a box culvert 5 feet square to support a road roller weighing 12 tons (*Class No. 1*), fill 2 feet deep.

Solution. The maximum load will occur when the rear wheel is at the center of the span, which is two-thirds of 12 tons, or 8 tons, Fig. 123. This will be distributed over an area of 1 foot by 9 feet 6 inches. The live load is, therefore, $8 \times 2{,}000 \div 9.5 = 1{,}664$ pounds for a strip 1 foot wide. The dead load will be $100 \times 2 = 200$ pounds per square foot for fill and, assuming that the top slab will be 8 inches thick, $12.5 \times 8 = 100$ pounds per square foot.

The moments will be as follows:

$$\text{Live load } M = \frac{Wl}{4} \times 12 = \frac{1{,}664 \times 5}{4} \times 12 \qquad = 24{,}960 \text{ in.-lb.}$$

$$\text{Add 35 per cent for impact } = 24{,}960 \times .35 = 8{,}736 \text{ in.-lb.}$$

$$\text{Dead load} = M = \frac{Wl^2}{8} \times 12 = \frac{300 \times 5^2}{8} \times 12 \quad = \underline{11{,}250} \text{ in.-lb.}$$

$$44{,}946 \text{ in.-lb.}$$

Placing this equal to $95\,b\,d^2$ where $b = 12$

$$95 \times 12 \times d^2 = 44{,}946.$$

$$d^2 = 39.43$$

$$d = 6.28$$

Make the total thickness 8 inches. The steel required equals $.00675 \times 6.28 = .04239$ square inch per inch of width. $\frac{3}{4}$-inch round bars spaced 10 inches on centers will fulfill the requirements.

The earth pressure on the sides is as follows:

At the top	$\frac{Wh}{3} = \frac{100 \times 3.2}{3} = 106$ lb. per sq. ft.
At the bottom	$= \frac{100 \times 7.2}{3} = 240$ lb. per sq. ft.
Average pressure	$(106 + 240) \div 2 = 173$ lb. per sq. ft.
Pressure for live load	$P = 1{,}664 \div 3 = 555$ lb. per sq. ft.
Total pressure	$173 + 555 = 728$ pounds

The bending moment for this load is

$$M = \frac{Wl^2}{8} \times 12 = \frac{728 \times 5^2}{8} \times 12 = 27{,}300 \text{ in.-lb.}$$

insure stirrups. Use $\frac{3}{8}$-inch round bars, spaced 9 inches on centers, Fig. 123. The bottom will be made 8 inches thick, also, and reinforced with $\frac{3}{4}$-inch round bars, spaced 10 inches on centers. Temperature bars must also be provided.

GIRDER BRIDGES

Method of Design. Girder bridges are being extensively used for country highways for spans from 20 to 40 feet. They are sometimes used for spans up to 60 feet and often for spans as short as 16 feet. Fig. 124 shows the section of one-half the

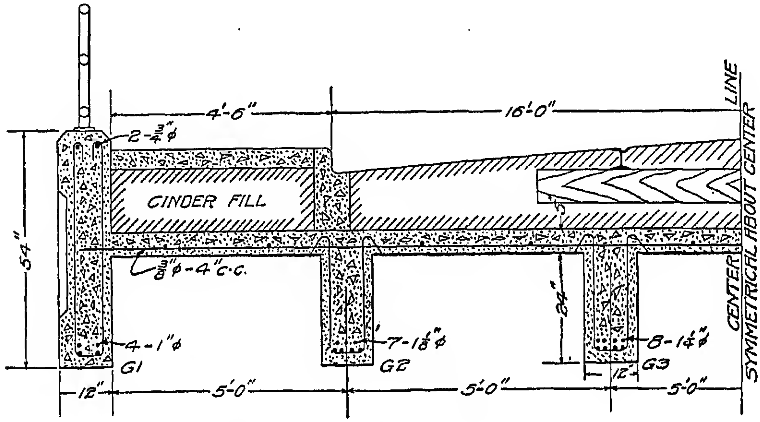


Fig. 124. Design Diagram for Girder Bridge

width of such a bridge. The slab of such a bridge must always be paved or macadamized so that no wheels will come direct on the concrete.

Illustrative Example. Design a girder bridge with a clear span of 26 feet; width of roadway 16 feet; and two sidewalks each 4 feet 6 inches wide. The loading for this bridge to be as specified for *Class No. 2,* the car line being in the center of the bridge, a fill of six inches to be placed under the ties with a macadam-surfaced roadway.

The slab for such a structure should never be less than 5 inches thick on account of concentrated loads and shear due to road rollers and other such loads. The slab will be designed for a live

load of 500 pounds per square foot. The slab load and moment, therefore, would be as follows:

Live load	$4 \times 1 \times 500 =$	2,000
Slab, 5 inches	$\frac{5}{12} \times 150 \times 4 =$	250
Fill, 20 inches	$1\frac{2}{3} \times 125 \times 4 =$	833
		3,083

$$M = \frac{3{,}100 \times 4 \times 12}{8} = 18{,}600$$

$$95\,b\,d^2 = 18{,}600$$

$$95 \times 12 \times d^2 = 18{,}600$$

$$d^2 = 16.3$$

$$d = 4$$

The steel area equals $.00675 \times 4 \times 12$, or .32 square inches per foot of width, which requires $\frac{3}{8}$-inch round bars, spaced 4 inches on centers.

The outside girder G_1 supports one-half of the sidewalk load, which is as follows:

Live load 125	$125 \times 2\frac{1}{4} \times 26$	= 7,313 lb.
Walk 4 in. thick	$50 \times 2\frac{1}{4} \times 26$	= 2,925 lb.
Cinder fill 15 in.	$60 \times 1\frac{1}{4} \times 2\frac{1}{4} \times 26$	= 4,388 lb.
Slab 5 in.	$60 \times 2\frac{1}{4} \times 26$	= 3,510 lb.
Girder 12×54 in.	$150 \times 4\frac{1}{2} \times 26 \times 1$	= 17,550 lb.
		35,686 lb.

$$M = \frac{35{,}686 \times 26 \times 12}{8} = 1{,}391{,}754 \text{ in.-lb.}$$

This moment placed equal to $95\,b\,d^2$, when $b = 12$, would only require a depth of 35 inches to the center of the steel, while the total depth of the beam is 54 inches. Therefore, make $b = 12$ and $d = 51$, and solve for the moment factor R.

$$12 \times \overline{51}^2 \times R = 1{,}505{,}400$$

$$R = 48$$

Referring to the diagram, Fig. 99, it is at once to be seen that when $R = 48$, the compression in the concrete will be low and that a percentage of steel of .005 is more than actually will be required. However, that amount will be used. $12 \times 51 \times .005 = 3.1$

traight and 2 turned up near the ends. The shear per square nch is small, but stirrups should be used.

Girder G_3 will next be designed. For this beam there are three ive loads to be considered and the girder will be designed to support he maximum one combined with the dead load. The three live

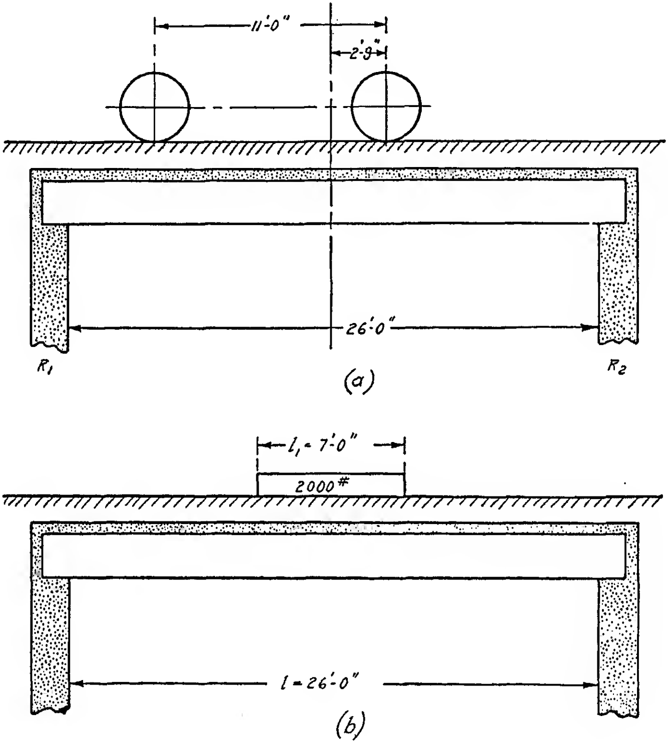


Fig. 125. Diagrams for Loadings for Road Roller and Electric Car

oads are: the uniform load of 125 pounds per square foot, a 0-ton road roller, and a 40-ton electric car.

The dead load and moment for this load will be as follows:

Macadam and fill	$1\frac{2}{3}\times125\times5\times26=$	27,084 lb.
Slab	$\frac{5}{12}\times150\times5\times26=$	8,125 lb.
Beam 12″×24″ (assumed)	$1\times2\times150\times26=$	7,800 lb.
		43,009 lb.

$$M=\frac{43,000\times26\times12}{8}=1,677,000 \text{ in. lb.}$$

foot would be $125 \times 5 \times 26 = 16{,}250$ pounds.

$$M = \frac{16{,}250 \times 26 \times 12}{8} = 633{,}750 \text{ in.-lb.}$$

Since the fill is so small the weight of a road roller or car cannot be distributed to any great extent by this means, it will not be considered in the calculations. Each of these beams may be required to support the whole weight of the front wheel and half the weight of the rear wheel. This moment will be a maximum when one wheel is one-fourth of the distance between the center of wheels from the center of the span of the bridge.

The maximum reaction is at the right and is

$$R_1 = \frac{13{,}333 \times 4.75}{26} + \frac{13{,}333 \times 15.75}{26} = 10{,}478$$

Then

$$M = 10{,}478 \times 10.25 \times 12 = 1{,}288{,}794 \text{ in.-lb.}$$

The maximum load produced on girders G_3 by an electric car takes place when one of the trucks is at the center of the span. Each of these girders at that time would be supporting one-fourth of the total weight of 40 tons, which is 10 tons, see Fig. 125.

The moment is, therefore

$$M = \left(\frac{20{,}000 \times 26}{4} - \frac{20{,}000 \times 7}{8}\right) 12 = 1{,}350{,}000$$

Add 35 per cent for impact	472,500
	1,822,500

The electric car produces a greater bending moment than either of the other live loads and, therefore, will be used together with the dead load. That is, $1{,}822{,}500 + 1{,}677{,}000 = 3{,}499{,}500$. Let d equal 25.5, then $25.5 \times .88 = 22.4$ inches. The required amount of steel then is $3{,}499{,}500 \div 22.4 \times 16{,}000 = 9.8$ square inches. Eight bars $1\frac{1}{4}$ inches in diameter will be used, one-half of which will be turned up in pairs at different points near the ends of the girder.

The shear in this girder will be $\frac{1}{2}(20{,}000 + 43{,}000) = 31{,}500$ pounds.

$$V = \frac{31{,}500}{12 \times 23} = 114 \text{ lb. per sq. in.}$$

Therefore stirrups must be used. They should be $\frac{3}{8}$ of an inch

in diameter, used throughout the length of the girder, and spaced not over 6 inches apart near the ends of the girders.

The bending moment for girder G_2 will be taken as the mean of girders G_1 and G_3, plus the dead load, which will be as follows:

$$G_1 = 1{,}505{,}400 \text{ in.-lb.}$$
$$G_3 = 3{,}499{,}500 \text{ in.-lb.}$$
$$1{,}505{,}400 + 3{,}499{,}500 = 5{,}004{,}900 \text{ in.-lb.}$$
$$G_2 = 5{,}004{,}900 \div 2 = 2{,}502{,}450 \text{ in.-lb.}$$

The steel required equals $2{,}502{,}450 \div 22.4 \times 16{,}000 = 7$ square inches. Seven bars $1\frac{1}{8}$ inches in diameter will be used, $\frac{3}{8}$ of which will be turned up near the ends of the girders. Use $\frac{3}{8}$-inch shear bars.

In designing girder bridges the designer must always investigate the shear in the girders and the compression in the T-beams very carefully and see that these stresses are satisfied.

Arch Culverts. Arch culverts come under the head of arches and as the general subject of arches, and especially the application of reinforced concrete to arch construction, is taken up in Part V, this subject will not be further discussed here.

COLUMNS

Methods of Reinforcement. The laws of mechanics, as well as experimental testing on full-sized columns of various structural materials, show that very short columns, or even those whose length is ten times their smallest diameter, will fail by crushing or shearing of the material, assuming that the line of pressure is practically coincident with the axis of the column. If the columns are very long, say twenty or more times their smallest diameter, they will probably fail by bending, which will produce an actual tension on the convex side of the column. The line of division between long and short columns is, practically, very uncertain, owing to the fact that the center line of pressure of a column is frequently more or less eccentric because of irregularity of the bearing surface at top or bottom. Such an eccentric action will cause buckling of the column, even when its length is not very great. On this account, it is always wise, especially for long columns, to place reinforcing bars within the column. The reinforcing bars consist of longitudinal bars

bands of small bars spaced from 6 to 18 inches apart vertically, which bind together the longitudinal bars. The longitudinal bars are used for the purpose of providing the necessary transverse strength to prevent buckling of the column. As it is practically impossible to develop a satisfactory theory on which to compute the required tensional strength in the convex side of a column of given length, without making assumptions which are themselves of doubtful accuracy, no exact rules for the sizes of the longitudinal bars required to resist buckling in a column will be given. The bars ordinarily used vary from $\frac{1}{2}$ inch square to 1 inch square; and the number is usually four, unless the column is very large—400 square inches or larger—or is rectangular rather than square. It has been claimed by many, that longitudinal bars in a column may actually be a source of danger, since the buckling of the bars outward may tend to disintegrate the column. This buckling can be avoided, and the bars made mutually self-supporting, by means of the bands which are placed around the column. These bands are usually $\frac{1}{4}$-inch or $\frac{3}{8}$-inch round or square bars. The specifications of the Prussian Public Works for 1904 require that these horizontal bars shall be spaced a distance not more than 30 times their diameter, which would be $7\frac{1}{2}$ inches for $\frac{1}{4}$-inch bars, and $11\frac{1}{4}$ inches for $\frac{3}{8}$-inch bars. The bands in the column are likewise useful to resist the bursting tendency of the column, especially when it is short. They will also reinforce the column against the tendency to shear, which is the method by which failure usually takes place. The angle between this plane of rupture and a plane perpendicular to the line of stress is stated to be 60°. If, therefore, the bands are placed at a distance apart equal to the smallest diameter of the column, any probable plane of rupture will intersect one of the bands, even if the angle of rupture is somewhat smaller than 60°.

The following specifications are from the code for Greater New York (1912):

27. Axial compression in columns without hoops, bands, or spirals, and with not less than $\frac{1}{2}$ nor more than 4 per cent of vertical reinforcement secured against lateral displacement by steel ties placed not farther apart than 15 diameters of the rods nor more than 12 in., shall not exceed 500 lb. per sq. in. on the concrete nor 6,000 lb. per sq. in. on the vertical reinforcement.

and in no case more than three inches, and with not less than 1 nor more than 4 per cent of vertical reinforcement, shall not exceed 725 lb. per sq. in. on the concrete within the hoops or spirals nor 8,700 lb. per sq. in. on the vertical reinforcement.

Design of Columns. It may be demonstrated by theoretical mechanics, that if a load is jointly supported by two kinds of material with dissimilar elasticities, the proportion of the loading borne by each will be in a ratio depending on their relative areas and moduli of elasticity. The formula for this may be developed as follows:

C = Total unit compression upon concrete and steel in pounds per square inch = total load divided by the combined area of the concrete and the steel
c = Unit compression in the concrete, in pounds per square inch
s = Unit compression in the steel, in pounds per square inch
p = Ratio of area of steel to total area of column
$n = \frac{E_s}{E_c}$ = ratio of the moduli of elasticity
ϵ_s = Deformation per unit of length in the steel
ϵ = Deformation per unit of length in the concrete
A_s = Area of steel
A_c = Area of concrete

The total compressive force in the concrete equals $A_c \times c$; and that in the steel equals $A_s \times s$.

The sum of these compressions equal the total compression; and therefore

$$C(A_c + A_s) = A_c\, c + A_s s$$

The actual lineal compression of the concrete equals that of the steel; therefore

$$\frac{c}{E_c} = \frac{s}{E_s}$$

From this equation, since $n = \frac{E_s}{E_c}$, we may write the equation $nc = s$

Solving the above equation for C, we obtain

$$C = \frac{A_c c + A_s s}{A_c + A_s}$$

Substituting the value of $s = nc$, we have

$$C = c\left(\frac{A_c + A_s n}{\quad}\right) = c\left(\frac{A_s + A_c - A_s + A_s n}{\quad}\right)$$

Substituting this value of $\frac{A_s}{A_c+A_s}$ in the above equation, we have

$$C=c\,(1-p+p\,n)$$

Solving this equation for p, we obtain

$$p=\frac{C-c}{c\,(n-1)} \tag{55}$$

Examples. 1. A column is designed to carry a load of 160,000 pounds. If the column is made 16 inches square, and the load per square inch to be carried by the concrete is limited to 500 pounds, what must be the ratio of the steel and how much steel would be required?

Solution. A column 16 inches square has an area of 256 square inches. Dividing 160,000 by 256, we have 625 pounds per square inch as the total unit compression upon the concrete and the steel, which is C in the above formula. Assume that the concrete is 1:2:4 concrete, and that the ratio of the moduli of elasticity n is, therefore, 15. Substituting these values in Equation (55), we have

$$p=\frac{625-500}{500\,(15-1)}=.01786$$

Multiplying this ratio by the total area of the column—256 square inches—we have 4.57 square inches of steel required in the column. This would be amply provided by 4 bars $1\frac{1}{8}$ inches square. The bands, if made of $\frac{1}{2}$-inch bars, should be spaced not more than $7\frac{1}{2}$ inches (15 diameters) apart.

2. A column 16 inches square is subjected to a load of 126,000 pounds and is reinforced by four $\frac{7}{8}$-inch square bars besides the bands. What is the actual compressive stress in the concrete per square inch, assuming the same grade of concrete as above?

Solution. Dividing the total stress, 126,000, by the area, 256, we have the combined unit stress $C=492$ pounds per square inch. By inverting one of the equations above, we have

$$c=\frac{C}{1-p+n\,p}$$

In the above case, the four $\frac{7}{8}$-inch bars have an area of 3.06 square inches; and therefore

$$p=\frac{3.06}{256}=.012 \qquad n=15$$

Substituting these values in the above equations, we have

$$c=\frac{492}{1-.012+(.012\times 15)}=\frac{492}{1.168}=421 \text{ lb. per sq. in.}$$

The net area of the concrete in the above problem is 252.94 square inches. Multiplying this by 421, we have the total load carried by

the concrete, which is 106,488 pounds. Subtracting this from 126,000 pounds, the total load, we have 19,512 pounds as the compressive stress carried by the steel. Dividing this by 3.06, the area of the steel, we have 6,376 pounds as the unit compressive stress in the steel. This is practically fifteen times the unit compression in the concrete, which is an illustration of the fact that if the compression is shared by the two materials in the ratio of their moduli of elasticity, the unit stresses in the materials will be in the same ratio. This unit stress in the steel is about four-tenths of the working stress which may properly be placed on the steel. It shows that we cannot economically use the steel in order to reduce the area of the concrete, and that the chief object in using steel in the columns is in order to protect the columns against buckling, and also to increase their strength by the use of bands.

It sometimes happens that in a building designed to be structurally of reinforced concrete, the column loads in the columns of the lower story may be so very great that concrete columns of sufficient size would take up more space than it is desirable to spare for such a purpose. For example, it might be required to support a load of 320,000 pounds on a column 15 inches square. If the concrete (1:2:4) is limited to a compressive stress of 500 pounds per square inch, we may solve for the area of steel required, precisely as was done in Example 1. We should find that the required percentage of steel was 13.17 per cent, and that the required area of the steel was, therefore, 29.6 square inches. But such an area of steel could carry the entire load of 320,000 pounds without the aid of the concrete, and would have a compressive unit stress of only 10,800 pounds. In such a case, it would be more economical to design a steel column to carry the entire load, and then to surround the column with sufficient concrete to fireproof it thoroughly. Since the stress in the steel and the concrete are divided in proportion to their relative moduli of elasticity, which is usually about 12 to 15, we cannot develop a working stress of say 16,000 pounds per square inch in the steel without at the same time developing a compressive stress of 1,100 to 1,300 pounds in the concrete, which is objectionably high as a working stress.

Hooped Columns. It has been found that the strength of a

the column by numerous hoops or bands or by a spiral of steel. The basic principle of this strength can best be appreciated by considering a section of stovepipe filled with sand and acting as a column. The sand alone, considered as a column, would not be able to maintain its form, much less to support a load, especially if it were dry. But when it is confined in the pipe, the columnar strength is very considerable. Concrete not only has great crushing strength, even when plain, but can also be greatly strengthened against failure by the tensile strength of bands which confine it. The theory of the amount of this added resistance is very complex, and will not be given here. The general conclusions, in which experimental results support the theory, are as follows:

1. The deformation of a hooped column is practically the same as that of a plain concrete column of equal size for loads up to the maximum for a plain column.
2. Further loading of a hooped column still further increases the shortening and swelling of the column, the bands stretching out, but without causing any apparent failure of the column.
3. Ultimate failure occurs when the bands break, or, having passed their elastic limit, stretch excessively.

Hooped columns may thus be trusted to carry a far greater unit load than plain columns, or even columns with longitudinal rods and a few bands. There is one characteristic that is especially useful for a column which is at all liable to be loaded with a greater load than its nominal loading. A hooped column will shorten and swell very perceptibly before it is in danger of sudden failure, and will thus give ample warning of an overload.

Considère has developed an empirical formula based on actual tests, for the strength of hooped columns, as follows:

$$\text{Ultimate strength} = c'A + 2.4\,s'pA \qquad \textbf{(56)}$$

where c' is ultimate strength of the concrete; s' is elastic limit of the steel; p is ratio of area of the steel to the whole area; and A is whole area of the column. This formula is applicable only for reinforcement of mild steel. Applying this formula to a hooped column tested to destruction by Professor Talbot, in which the ultimate strength c' of similar concrete was 1,380 pounds per square inch, the elastic limit s' of the steel was 48,000 pounds per square inch; the ratio p of reinforcement was .0212; and the area A was 104

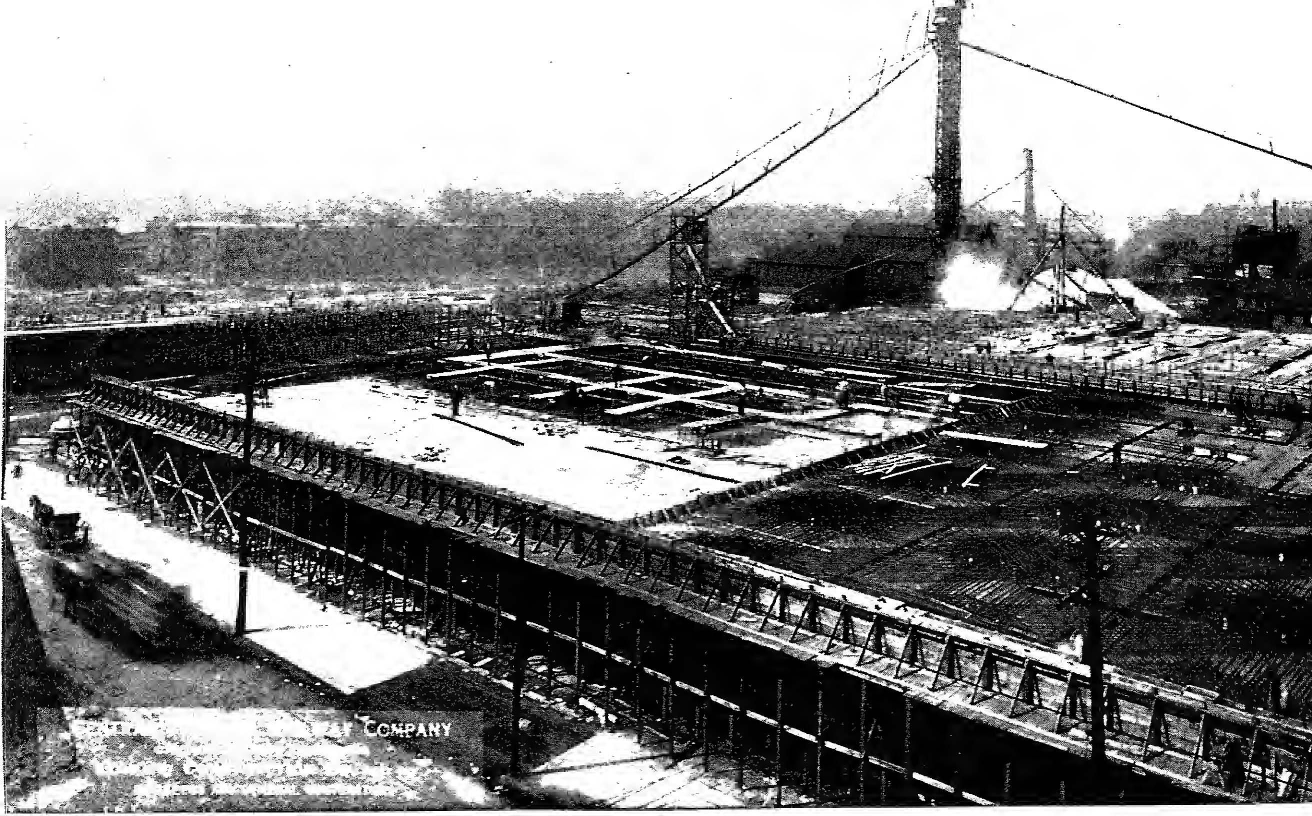
have, for the computed ultimate strength, 409,900 pounds. The actual ultimate by Talbot's test was 351,000 pounds, or about 86 per cent.

Talbot has suggested the following formulas for the ultimate strength of hooped columns per square inch:

$$\text{Ultimate strength} = 1{,}600 + 65{,}000\,p \text{ (for mild steel)} \quad (57)$$

$$\text{Ultimate strength} = 1{,}600 + 100{,}000\,p \text{ (for high steel)} \quad (58)$$

In these formulas p applies only to the area of concrete within the hooping; and this is unquestionably the correct principle, as the concrete outside of the hooping should be considered merely as fire protection and ignored in the numerical calculations, just as the concrete below the reinforcing steel of a beam is ignored in calculating the strength of the beam. The ratio of the area of the steel is computed by computing the area of an equivalent thin cylinder of steel which would contain as much steel as that actually used in the bands or spirals. For example, suppose that the spiral reinforcement consisted of a $\frac{1}{2}$-inch round rod, the spiral having a pitch of 3 inches. A $\frac{1}{2}$-inch round rod has an area of .196 square inch. That area for 3 inches in height would be the equivalent of a solid band .0653 inch thick. If the spiral had a diameter of, say, 11 inches, its circumference would be 34.56 inches, and the area of metal in a horizontal section, would be $34.56 \times .0653$, or 2.257 square inches. The area of the concrete within the spiral is 95.0 square inches. The value of p is therefore $2.257 \div 95.0 = .0237$. If the $\frac{1}{2}$-inch bar were made of high-carbon steel, the *ultimate* strength per square inch of the column would be $1{,}600 + (100{,}000 \times .0237) = 1{,}600 + 2{,}370$, or 3,970. The unit strength is considerably more than doubled. The ultimate strength of the whole column is, therefore, $95 \times 3{,}970$, or 377,150 pounds. Such a column could be safely loaded with about 94,300 pounds, provided its length were not so great that there was danger of buckling. In such a case, the unit stress should be reduced according to the usual ratios for long columns, or the column should be



CHARACTERISTIC REINFORCED CONCRETE CONSTRUCTION SHOWING REINFORCING STEEL IN POSITION
Chicago freight terminal for the "Soo" line covering eleven city blocks, or 18 acres of ground
Courtesy of Leonard Construction Company, General Contractors

of the column. The theoretical demonstration of the amount of this eccentricity depends on assumptions which may or may not be found in practice. The following formula is given without proof or demonstration, in Taylor and Thompson's treatise on Concrete:

$$f' = f\left(1 + \frac{6e}{b}\right) \qquad \textbf{(59)}$$

in which e is eccentricity of load; b is breadth of column; f is average unit pressure; f' is total unit pressure of outer fiber nearest to line of vertical pressure.

As an illustration of this formula, if the eccentricity on a 12-inch column were 2 inches, we should have b equals 12, and e equals 2. Substituting these values in Equation (**59**), we should have f' equals $2f$, which means that the maximum pressure would equal twice the average pressure. In the extreme case, where the line of pressure came to the outside of the column, or when e equals $\frac{1}{2}b$, we should have a maximum pressure on the edge of the column equal to four times the average pressure.

Any refinements in such a calculation, however, are frequently overshadowed by the uncertainty of the actual location of the center of pressure. A column which supports two equally loaded beams on each side is probably loaded more symmetrically than a column which supports merely the end of a beam on one side of it. The best that can be done is arbitrarily to lower the unit stress on a column that is probably loaded somewhat eccentrically.

TANKS

Design. The extreme durability of reinforced-concrete tanks and their immunity from deterioration by rust, which so quickly destroys steel tanks, have resulted in the construction of a large and increasing number of tanks in reinforced concrete. Such tanks must be designed to withstand the bursting pressure of the water. If they are very high compared with their diameter, it is even possible that failure might result from excessive wind pressure. The method of designing one of these tanks may best be considered from an example.

Illustrative Example. Suppose that it is required to design a reinforced-concrete tank with an inside diameter of 18 feet and with

a capacity of 50,000 gallons. At 7.48 gallons per cubic foot, a capacity of 50,000 gallons will require 6,684 cubic feet. If the inside diameter of the tank is to be 18 feet, then the 18-foot circle will contain an area of 254.5 square feet. The depth of the water in the tank will, therefore, be 26.26 feet. The lowest foot of the tank will therefore be subjected to a bursting pressure due to 25.26 vertical feet of water. Since the water pressure per square foot increases $62\frac{1}{2}$ pounds for each foot of depth, we shall have a total pressure of 1,610 pounds per square foot on the lowest foot of the tank. Since the diameter is 18 feet, the bursting pressure it must resist on each side is $\frac{1}{2}(18 \times 1{,}610)$, or 14,490 pounds. If we allow a working stress of 15,000 pounds per square inch, this will require .966 square inch of metal in the lower foot. Since the bursting pressure is strictly proportional to the depth of the water, we need only divide this number proportionally to the depth to obtain the bursting pressure at other depths. For example, the ring one foot high, at one-half the depth of the tank, should have .483 square inch of metal; and that at one-third of the depth should have .322 square inch of metal. The actual bars required for the lowest foot may be figured as follows: .966 square inch per foot equals .0805 square inch per inch; $\frac{3}{4}$-inch square bars, having an area .5625 square inch, will furnish the required strength when spaced 7 inches apart. At one-half the height, the required metal per lineal inch of height is half of the above, or .040. This could be provided by using $\frac{3}{4}$-inch bars spaced 14 inches apart; but this is not so good a distribution of metal as to use $\frac{5}{8}$-inch square bars having an area of .39 square inch, and to space the bars nearly 10 inches apart. It would give a still better distribution of metal to use $\frac{1}{2}$-inch bars spaced 6 inches apart at this point, although the $\frac{1}{2}$-inch bars are a little more expensive per pound, and, if they are spaced very closely, will add slightly to the cost of placing the steel. The size and spacing of bars for other points in the height can be similarly determined.

A circle 18 feet in diameter has a circumference of somewhat over 56 feet. Assuming, as a preliminary figure, that the tank is to be 10 inches thick at the bottom, the mean diameter of the base ring would be 18.83 feet, which would give a circumference of over 59 feet. Allowing a lap of 3 feet on the bars, this would require

to have bars rolled of this length, they are very difficult to handle, and require to be transported on the railroads on two flat cars. It is therefore preferable to use bars of somewhat more than half this length, say 32 feet 6 inches, and to make two joints in each band.

The bands which are used for ordinary wooden tanks are usually fastened at the ends by screw bolts. Some form of joint, which is as strong as the bar, should be used. It has been found that if deformed bars are overlapped from 18 inches to 3 feet, according to their size, and are then wired together tightly so that their lugs interlock, it will require a greater force than the strength of the bar to pull the joints apart after they are once thoroughly incased in the concrete and the concrete has hardened.

Test for Overturning. Since the computed depth of the water is over 26 feet, we must calculate that the tank will be, say 28 feet high. Its outer diameter will be approximately 20 feet. The total area exposed to the surface of the wind will be 560 square feet. We may assume that the wind has an average pressure of 50 pounds per square foot; but, owing to the circular form of the tank, we shall assume that its effective pressure is only one-half of this; and therefore, we may figure that the total overturning pressure of the wind equals 560×25, or 14,000 pounds. If this is considered to be applied at a point 14 feet above the ground, we have an overturning moment of 196,000 foot-pounds, or 2,352,000 inch-pounds.

Although it is not strictly accurate to consider the moment of inertia of this circular section of the tank as it would be done if it were a strictly homogeneous material, since the neutral axis, instead of being at the center of the section will be nearer to the compression side of the section, our simplest method of making such a calculation is to assume that the simple theory applies, and then to use a generous factor of safety. The effect of shifting the neutral axis from the center toward the compression side will be to increase the unit compression on the concrete and reduce the unit tension in the steel; but, as will be seen, it is generally necessary to make the concrete so thick that its unit compressive stress is at a very safe figure, while the reduction of the unit tension in the steel is merely on the side of safety.

Applying the usual theory, we have, for the moment of inertia of a ring section, $.049\ (d_1^4 - d^4)$. Let us assume as a preliminary

moment due to wind pressure

$$M=\frac{c\,I}{\frac{1}{2}\,d_1}=\frac{c\times45{,}337{,}842}{\frac{1}{2}\,d_1}=2{,}352{,}000\text{ in.-lb.}$$

in which $\frac{1}{2}\,d_1=118$ inches.

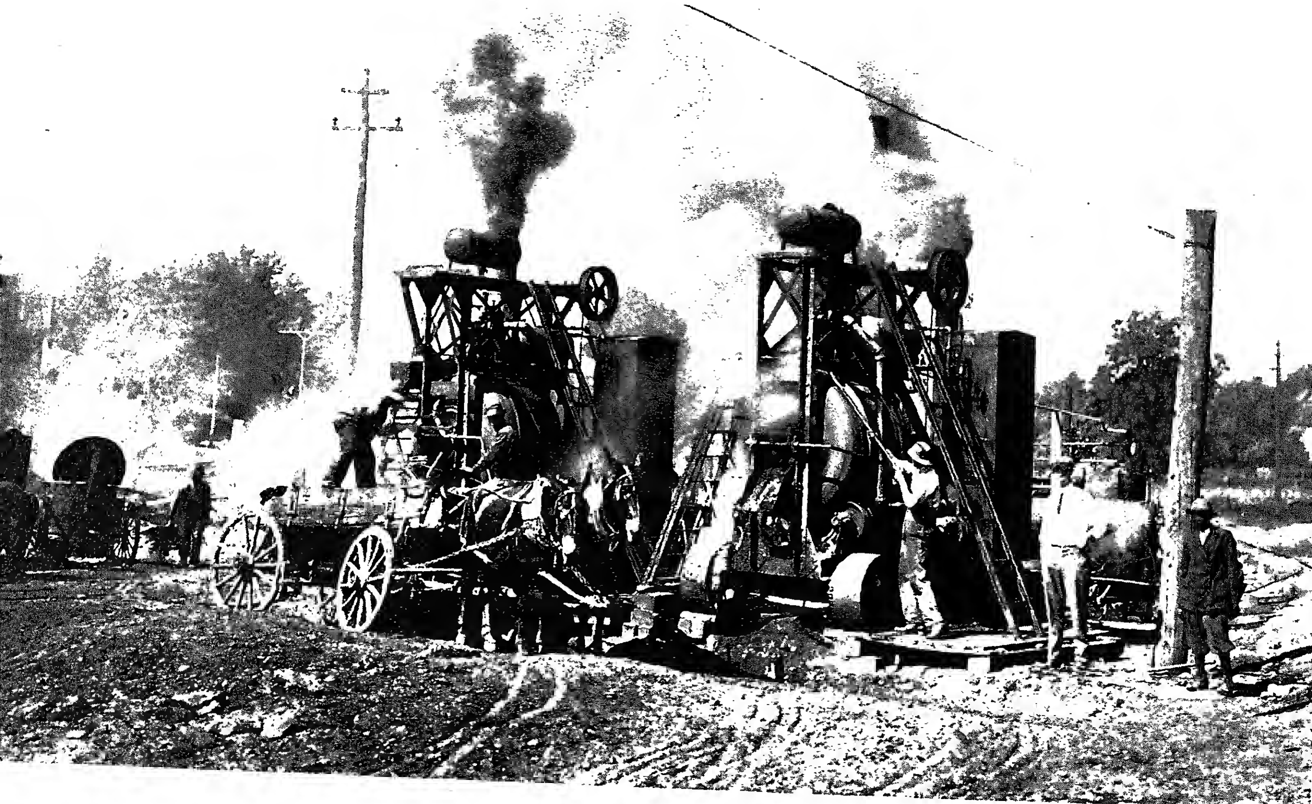
Solving the above equation for c, we have c equals a fraction less than 6 pounds per square inch. This pressure is so utterly insignificant, that, even if we double or treble it to allow for the shifting of the neutral axis from the center, and also double or treble the allowance made for wind pressure, although the pressure chosen is usually considered ample, we shall still find that there is practically no danger that the tank will fail owing to a crushing of the concrete due to wind pressure.

The above method of computation has its value in estimating the amount of steel required for vertical reinforcement. On the basis of 6 pounds per square inch, a sector with an average width of 1 inch and a diametral thickness of 10 inches would sustain a compression of about 60 pounds. Since we have been figuring working stresses, we shall figure a working tension of, say 16,000 pounds per square inch in the steel. This tension would therefore require $\frac{60}{16{,}000}$, or .0037 square inch of metal per inch of width. Even if $\frac{1}{4}$-inch bars were used for the vertical reinforcement, they would need to be spaced only about 17 inches apart. This, however, is on the basis that the neutral axis is at the center of the section, which is known to be inaccurate.

A theoretical demonstration of the position of the neutral axis for such a section is so exceedingly complicated that it will not be considered here. The theoretical amount of steel required is always less than that computed by the above approximate method; but the necessity for preventing cracks, which would cause leakage, would demand more vertical reinforcement than would be required by wind pressure alone.

Practical Details of Above Design. It was assumed as an approximate figure, that the thickness of the concrete side wall at

the base of the tank should be 10 inches. The calculations have shown that, so far as wind pressure is concerned, such a thickness is very much greater than is required for this purpose; but it will not do to reduce the thickness in accordance with the apparent requirements for wind pressure. Although the thickness at the bottom might be reduced below 10 inches, it probably would not be wise to make such reduction. It may, however, be tapered slightly towards the top, so that at the top the thickness will not be greater than 6 inches, or perhaps even 5 inches. The vertical bars in the lower part of the side wall must be bent so as to run into the base slab of tank. This will bind the side wall to the bottom. The necessity for reinforcement in the bottom of the tank depends very largely upon the nature of the foundation, and also, to some extent, on the necessity for providing against temperature cracks, as has been discussed on preceding pages. Even if the tank is placed on a firm and absolutely unyielding foundation, some reinforcement should be used in the bottom in order to prevent cracks which might produce leakage. These bars should run from a point near the center and be bent upward at least 2 or 3 feet into the vertical wall. Sometimes a gridiron of bars running in both directions is used for this purpose. This method is really preferable to the radial method.



KOEHRING HOT MIXERS IN OPERATION
Courtesy of Koehring Machine Company, Milwaukee, Wisconsin

MASONRY AND REINFORCED CONCRETE

PART IV

CONCRETE CONSTRUCTION WORK

MACHINERY FOR CONCRETE WORK

Concrete Plant. No general rule can be given for laying out a plant for concrete work. Every job is, generally, a problem by itself, and usually requires a careful analysis to secure the most economical results. Since it is much easier and cheaper to handle the cement, sand, and stone before they are mixed, the mixing should be done as near the point of installation as possible. All facilities for handling these materials, charging the mixer, and distributing the concrete after it is mixed must be secured and maintained. The charging and distributing are often done by wheelbarrows or carts; and economy of operation depends largely upon system and regularity of operation. Simple cycles of operations, the maintenance of proper runways, together with clocklike regularity, are necessary for economy. To shorten the distance of wheeling the concrete, it is very often found, on large buildings, that it is more economical to have two medium-sized plants located some distance apart, than to have one large plant. In city work, where it is usually impossible to locate the hoist outside of the building, it is constructed in the elevator shaft or light well. In purchasing a new plant, care must be exercised in selecting machinery that will not only be satisfactory for the first job, but that will fulfill the general needs of the purchaser on other work. All parts of the plant, as well as all parts of any one machine, should be easy to duplicate from stock, so that there will not be any great delay from breakdowns or from the use of worn-out parts.

The design of a plant for handling the material and concrete,

amount of concrete to be mixed per day, and the total amount required on the contract. It is very evident that on large jobs it pays to invest a large sum in machinery to reduce the number of men and horses; but, if not over 50 cubic yards are to be deposited per day, the cost of the machinery is a big item, and hand labor is generally cheaper. The interest on the plant must be charged against the number of cubic yards of concrete; that is, the interest on the plant for a year must be charged to the number of cubic yards of concrete laid in a year. The depreciation of the plant is found by taking the cost of the entire plant when new, and then appraising it after the contract is finished, and dividing the difference by the total cubic yards of concrete laid. This will give the depreciation per cubic yard of concrete manufactured.

CONCRETE MIXERS

Characteristics. The best concrete mixer is the one that turns out the maximum of thoroughly mixed concrete at the minimum of cost for power, interest, and maintenance. The type of mixer with a complicated motion gives better and quicker results than one with a simpler motion. There are two general classes of concrete mixers—*continuous* mixers and *batch* mixers. A continuous mixer is one into which the materials are fed constantly, and from which the concrete is discharged constantly. Batch mixers are constructed to receive the cement with its proportionate amount of sand and stone, all at one charge, and, when mixed, discharge it in a mass. No very distinct line can be drawn between these two classes, for many of these mixers are adapted to either continuous or batch mixing. Usually, batch mixers are preferred, as it is a very difficult matter to feed the mixers uniformly unless the materials are mechanically measured.

Continuous mixers usually consist of a long screw or pug mill that pushes the materials along a drum until they are discharged in a continuous stream of concrete. Where the mixers are fed with automatic measuring devices, the concrete is not regular, as there is no reciprocating motion of the materials. In a paper read before the Association of American Portland Cement Manufacturers, S. B.

of the material is mixed the required time, and then discharged, are the only type which will be found effective.

Concrete mixers use one of three different methods of combining the ingredients: the gravity, the rotary, or the paddle principle.

Gravity Mixers. Gravity mixers are the oldest type of concrete mixers. They require no power, the materials being mixed by striking obstructions which throw them together in their descent through the machine. These mixers are of simple construction comprising a steel trough or chute in which are contained the mixing members, consisting of pins or blades. The mixer is portable, and requires no skilled labor to operate it. There is nothing to get out of order or cause delays. It is adapted for both large and small jobs. In the former case, it is usually fed by measure, and by this method will produce concrete as fast as the materials can be fed to their respective bins and the mixed concrete can be taken from the discharge end of the mixer. On very small jobs, the best way to operate is to measure the batch in layers of stone, sand, and cement, respectively, men with shovels feeding them to the mixer.

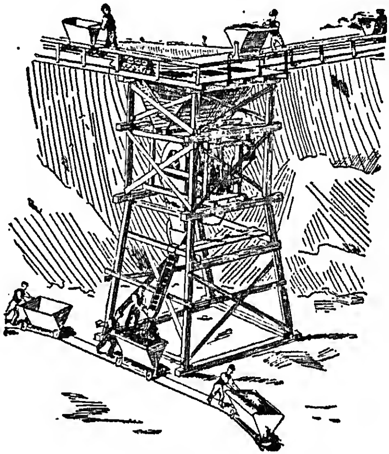


Fig. 126. Operation of Portable Gravity Mixer

There are two spray pipes placed on the mixer: for feeding by hand, one spray, only, would be used; the other spray is intended for use only when operating with the measure and feeder, and a large amount of water is required. These sprays are operated by handles which control two gate valves and regulate the quantity of water flowing from the spray pipes.

These mixers are made in two styles, sectional and non-sectional.

Rotary Mixers. *Cube Type.* The cube mixer shown in Fig. 127 consists of a cubical box of steel, at diagonally opposite corners of which hollow trunnions are provided which ride on rollers and support the drum. These trunnions are made large enough to serve as openings for charging and discharging the mixer. To rotate the cube, a circumferential rack is fastened around the drum, at right-angles to, and midway between, the hollow trunnions. This rack is

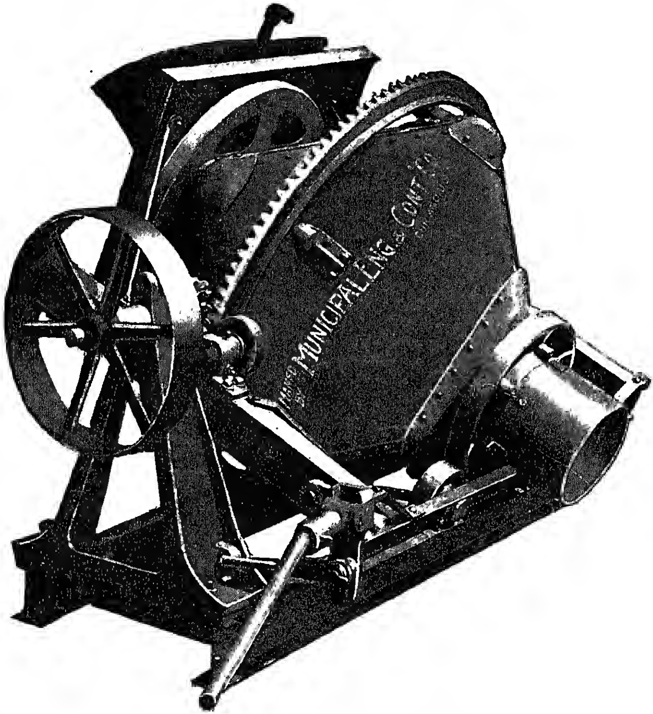


Fig. 127. Austin Improved Concrete Mixer
Courtesy of Municipal Engineering and Contracting Company, Chicago, Illinois

in mesh with a pinion shaft which is driven by the engine or motor. To discharge the mixer, an automatic dumping device is manipulated by the engine operator. At the charging end the usual form of hopper is provided. There are no paddles or blades of any kind to assist in the mixing, the stirring and kneeding of the cement being brought about by the tumbling action of the rotating cube.

Smith Type. Rotating mixers which contain reflectors or blades,

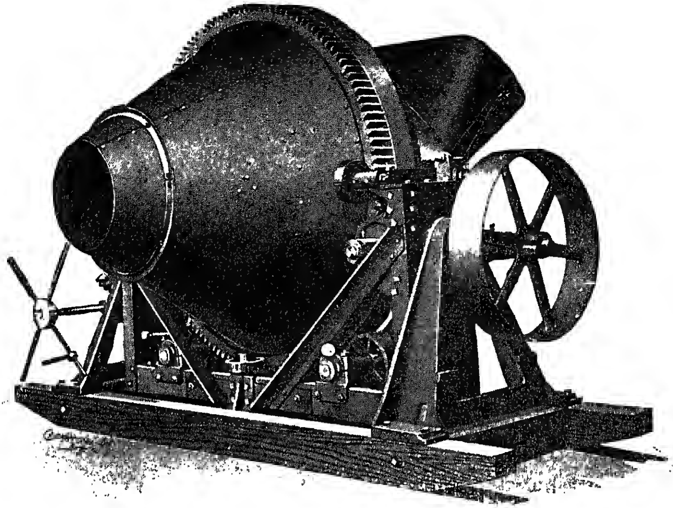


Fig. 128. Smith Mixer on Skids with Driving Pulley
Courtesy of T. L. Smith Company, Milwaukee, Wisconsin

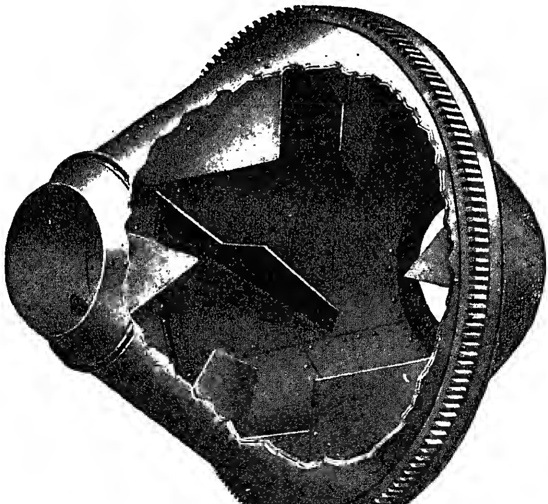


Fig. 129. Interior of Smith Mixer Drum
Courtesy of T. L. Smith Company, Milwaukee, Wisconsin

to side. Many of these machines can be filled and dumped while running, either by tilting or by their chutes. Fig. 128 illustrates the Smith mixer, and Fig. 129 gives a sectional view of the drum, and shows the arrangement of the blades. This mixer is furnished on skids with driving pulley. The concrete is discharged by tilting the drum, which is done by power.

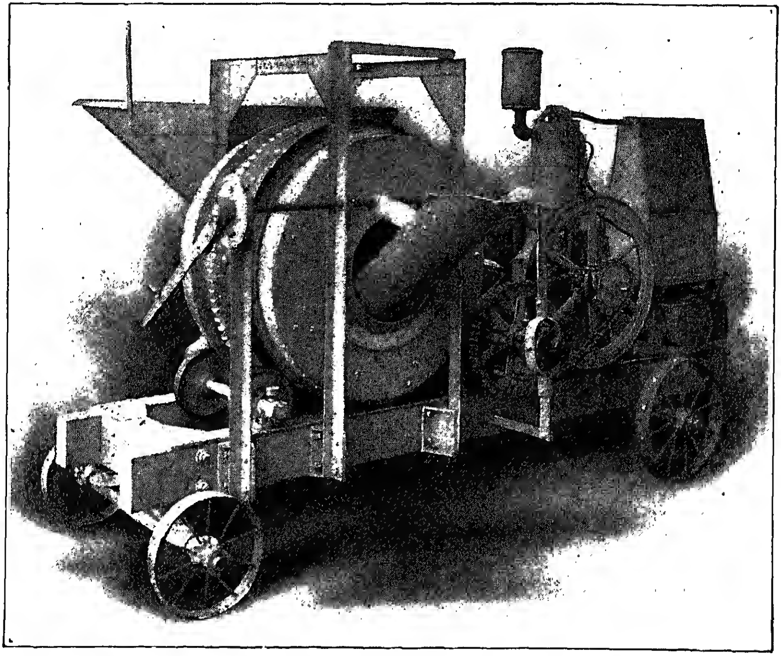


Fig. 130. Ransome Gasoline-Driven Concrete Mixing Outfit with Fixed Batch Hopper. Discharge Chute in Position for Mixing
Courtesy of Ransome Concrete Machinery Company, Chicago, Illinois

Ransome Type. Fig. 130 represents a Ransome mixer, which is a batch mixer. The concrete is discharged after it is mixed, without tilting the body of the mixer. It revolves continuously even while the concrete is being discharged. Riveted to the inside of the drum are a number of steel scoops or blades. These scoops pick up the material in the bottom of the mixer, and, as the mixer revolves, carry the material upward until it slides out of the scoops, which, therefore, assist in mixing the materials.

Smith-Chicago Type. The Smith-Chicago mixer, like the Ransome, does not tilt its drum when discharging the concrete. Discharge is accomplished by placing the chute in the position shown in Fig. 131. The outfit shown consists of the mixer, steam engine, boiler, power charger, and water tank mounted on a steel truck.

Paddle Mixers. Paddle mixers may be either continuous or of the batch type. Mixing paddles, on two shafts, revolve in opposite

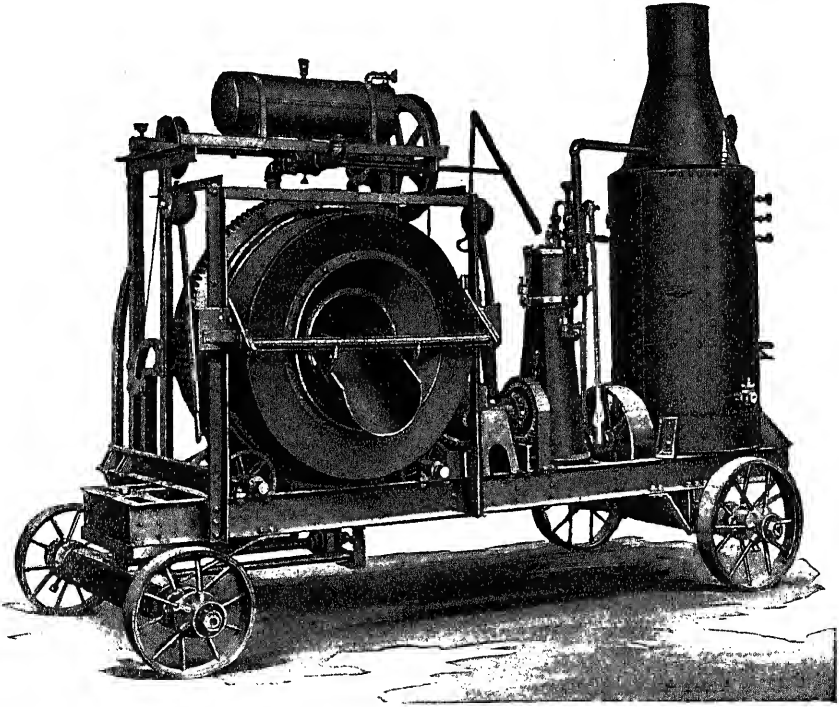


Fig. 131. Smith-Chicago Concrete Mixer on Steel Truck with Steam Engine and Boiler. Chute Shown in Discharging Position
Courtesy of T. L. Smith Company, Milwaukee, Wisconsin

directions, and the concrete falls through a trapdoor in the bottom of the machine. In the continuous type, the materials should be put in at the upper end so as to be partially mixed while dry. The water is supplied near the middle of the mixer. Fig. 132 represents a type of the paddle mixer.

Automatic Measurers for Concrete Materials. Mechanical measuring machines for concrete materials have not been very

uniform. If the machine is adjusted for sand with a certain percentage of moisture, and then is suddenly supplied with sand having greater or less moisture, the adjustment must be changed or the mixture will not be uniform. If the attendant does not watch the condition of the materials very closely, the proportions of the ingredients will vary greatly from what they should.

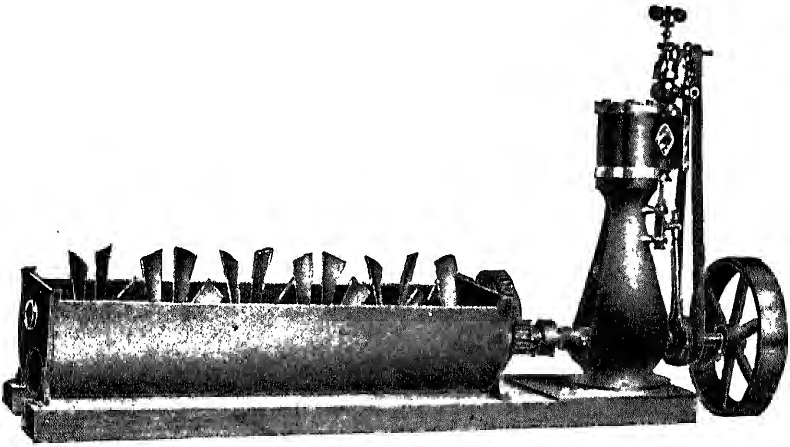


Fig. 132. Paddle Mixer

SOURCES OF POWER

General Considerations. In each case the source of power for operating the mixer, conveyors, hoists, derricks, or cableways must be considered. If it is possible to run the machinery by electricity, it is generally economical to do so. But this will depend a great deal upon the local price of electricity. When all the machinery can be supplied with steam from one centrally located boiler, this arrangement will be found perhaps more efficient.

In the construction of some reinforced-concrete buildings, a part of the machinery was operated by steam and a part by electricity. In constructing the Ingalls Building, Cincinnati, the machinery was operated by a gas engine, an electric motor, and a steam engine. The mixer was generally run by a motor; but by shifting the belt, it could be run by the gas engine. The hoisting was done by a 20-horsepower

TABLE XXV
Dimensions for Ransome Steam Engines

No. of Mixer		1	2	3	4
Size of Batch		10 cu. ft.	20 cu. ft.	30 cu. ft.	40 cu. ft.
Capacity per Hour (Cu. Yds.)		10	20	30	40
Horsepower Required	Engine Rated	6 by 6 7 h.p.	7 by 7 10 h.p.	8 by 8 14 h.p.	9 by 9 20 h.p.
	Boiler Rated	30 by 72 10 h.p.	36 by 78 15 h.p.	36 by 96 20 h.p.	42 by 102 30 h.p.
Speed of Drum (Rev. per min.)		16	15	14½	14
Speed of Driving Shaft (Rev. per min.)		116	122	94	99

Lidgerwood engine. This engine was also connected up to a boom derrick, to hoist lumber and steel. The practice of operating the machinery of one plant by power from different sources is to be questioned; but the practice of operating the mixer by steam and the hoist by electricity seems to be very common in the construction of buildings. A contractor, before purchasing machinery for concrete work, should carefully investigate the different sources of power for operating the machinery, not forgetting to consider the local conditions as well as general conditions.

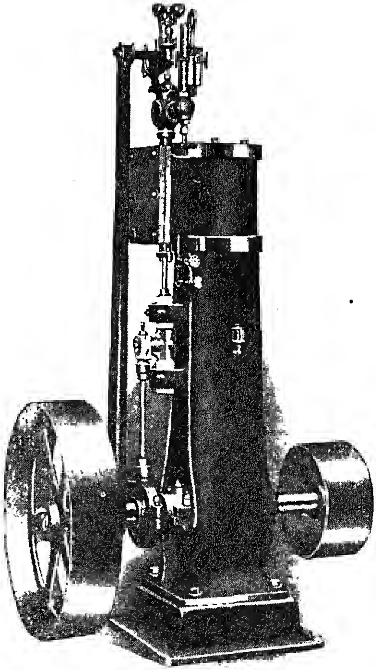


Fig. 133. Typical Steam Engine for Hoisting Purposes
Courtesy of Ransome Concrete Machinery

Power for Mixing Concrete. A vertical steam engine is generally used to operate the mixer.

mixers are mounted on the same frame; but, on account of the weight, it is necessary to mount the larger sizes on separate frames. Fig. 133 shows a Ransome disk crank, vertical engine, and Table XXV is taken from a Ransome catalogue on concrete machinery. These engines are well built, heavy in construction, and will stand hard work and high speed.

Gasoline Engines. Gasoline engines are used to some extent to operate concrete mixers. Their use, so far, has been limited chiefly to portable plants, such as are used for street work. The fuel for the gasoline engine is much easier moved from place to place than the fuel for a steam engine. Another advantage that the gasoline engine has over the steam engine is that it does not require the constant attention of an engineer.

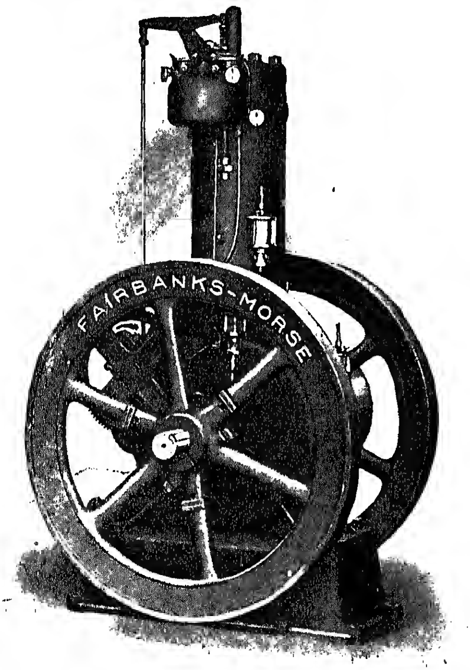


Fig. 134. Typical Single-Cylinder Gasoline Engine for Hoisting Purposes
Courtesy of Fairbanks, Morse & Company, Chicago

There are two types of engines—the *horizontal* and the *vertical*. The vertical engines occupy much less floor space for a given horsepower than the horizontal. While each type has its advantages and disadvantages, there does not really appear to be any very great advantages of one type over the other. Both types of engines are what are commonly known as *four-cycle* engines. In the operation of a 4-cycle engine, 4 strokes of the piston

about 1 gallon for each rated horsepower for any given size of engine. At 15 cents per gallon for gasoline, the hourly expense per horsepower will be 1.5 cents.

HOISTING AND TRANSPORTING EQUIPMENT

General Types of Units. When the concrete requires hoisting, it is done sometimes by the same engine that is used in mixing the concrete. It is generally considered better practice on large buildings to have a separate engine to do the hoisting. If it is possible to use a standard hoist, it is usually economical to do so. These hoists are equipped with automatic dump buckets.

Typical Hoisting Engine. Fig. 135 shows a standard double-cylinder, double-friction-drum hoisting engine of the Lambert type.

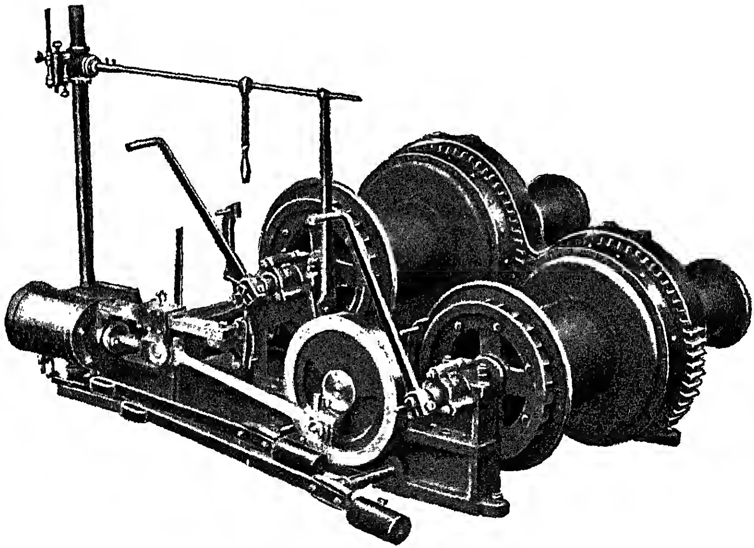


Fig. 135. Lambert Hoisting Engine

This type of engine is designed to fulfill the requirements of a general contractor for all classes of derrick work and hoisting. Steam can be applied by a single boiler, or from a boiler that supplies various engines with steam. The double-friction drums are independent of each other; therefore one or two derricks can be handled at the same time, if desired. This hoist is fitted with ratchets and pawls, and ... the end of each drum shaft. The winch

Horse-Power Usually Rated	Dimensions of Cylinders		Dimensions of Drums		Weight Hoisted Single Line (Pounds)	Suitable Weight for Pile-Driving Hammer for Quick Work (Pounds)
	Diameter (Inches)	Stroke (Inches)	Diameter (Inches)	Length between Flanges (Inches)		
10	$5\frac{1}{2}$	8	12	16	2,500	1,600
14	$6\frac{1}{2}$	8	12	16	3,000	2,000
20	7	10	14	18	5,000	3,000
25	$7\frac{1}{2}$	10	14	24	6,500	4,000
30	$8\frac{1}{2}$	10	14	24	8,000	5,000
35	9	10	14	24	9,000	5,000
40	$9\frac{1}{2}$	10	16	23	10,000	6,000

heads can be used for any hoisting or hauling desired, independent of the drums. These engines are also geared with reversible link motion.

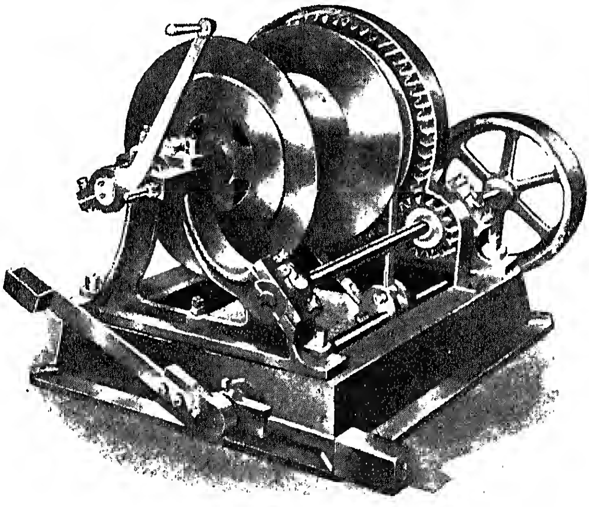


Fig. 136. Single Drum, Cone-Friction Belt Hoist.
Courtesy of Ransome Concrete Machinery Company, Chicago

The standard sizes and dimensions of Lambert hoisting engines are given in Table XXVI.

Cone-Friction Belt Hoist. A single drum cone-friction hoist of the Ransome type is illustrated in Fig. 136. The same engine that

drives the mixer can be used to operate the crab hoist. By means of a belt this hoist can be connected up to any engine and, when so connected, is ready for hoisting purposes. The hoisting drum is controlled by one lever. This hoist can be run by an electric motor, if desired.

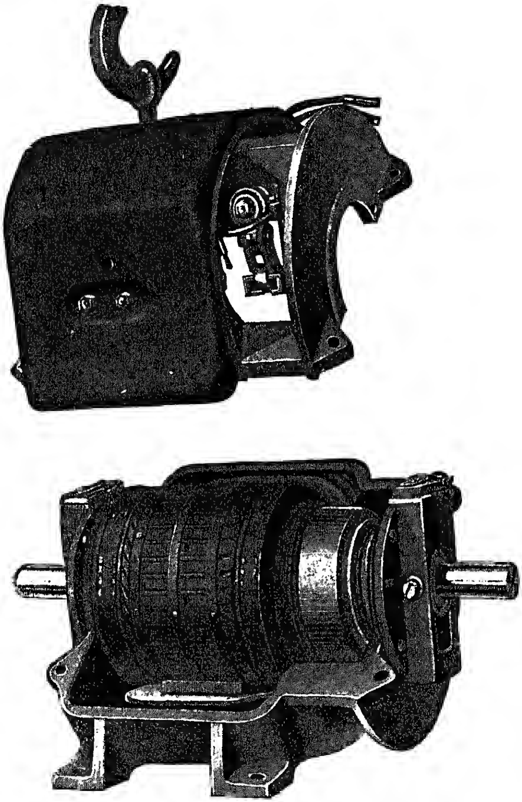


Fig. 137. Type "K" Hoisting Motor Showing Fields Parted
Courtesy of Westinghouse Electric and Manufacturing Company, East Pittsburgh. Pennsylvania

Electric Motors. Very often the cycle of operation of a hoist is of an intermittent character. The power required is at a maximum only a part of the time, even though the hoist may be operated practically continuously. From an economical point of view, these conditions give the electric-motor-driven hoist special advantages, in that the electric hoist would always be ready, but using power only

handled. The ease with which a motor is moved, and the simplicity of the connection to the service supply—requiring only two wires to be connected—are also in favor of the electric motor.

Fig. 137 shows a motor made by the Westinghouse Electric and Manufacturing Company, which is designed for the operation of cranes and hoists, or for intermittent service in which heavy starting torques and a wide speed variation are required. The frames are enclosed, to guard against dirt and moisture, but are so designed that the working parts may be exposed for inspection or adjustment without dismantling. These motors are series-wound, and are designed for operating on direct-current circuits. The motor frames are of cast steel, nearly square in section and very compact. The frame is built in two parts, and so divided that the upper half of the field can be removed without disturbing the gear or shaft, making it easy to take out a pole piece and field coils, or to remove the armature. Fig. 138 shows the controller for this type of motor. These controllers, when used for crane service, may be placed directly in the crane cage and operated by hand, or mounted on the resistance frames outside the cage, and operated by bell cranks and levers, so that the attendant may stand closer to the operating handles and away from the contacts and resistance.

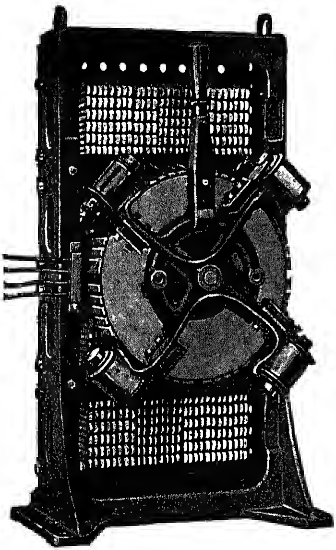


Fig. 138. Westinghouse Regulating and Reversing Control

Polyphase induction motors are being used to some extent for general hoisting and derrick work. These motors may be of the two-phase or three-phase type; but the latter are slightly more efficient. These motors are provided with resistances in the motor circuit, and with external contacts for varying the same. Two capacities of resistance can be furnished: (a) intermittent service,

In constructing large reinforced-concrete buildings, usually a separate hoist is used to elevate the steel and lumber for the forms. It may be equipped with either an electric motor or an engine, depending upon the general arrangement of the plant. These hoists are usually of the single-drum type.

Hoisting Buckets. In building construction, concrete is usually hoisted in automatic dumping buckets. The bucket is designed to slide up and down a light framework of timber, as shown in Fig. 139, and to dump automatically when it reaches the proper place to dump. The dumping of the buckets is accomplished by the bucket pitching forward at the point where the front guide in the hoisting tower is cut off. The bucket rights itself automatically as soon as it begins to descend. These buckets are often used for hoisting sand and stone as well as concrete. The capacity of the buckets varies from 10 cubic feet to 40 cubic feet. Fig. 140 shows a Ransome bucket which has been satisfactorily used for this purpose.

Methods of Charging Mixers. The mixers are usually charged by means of wheelbarrows, although other means are sometimes used. Fig. 141 shows the type of wheelbarrow customarily used for this

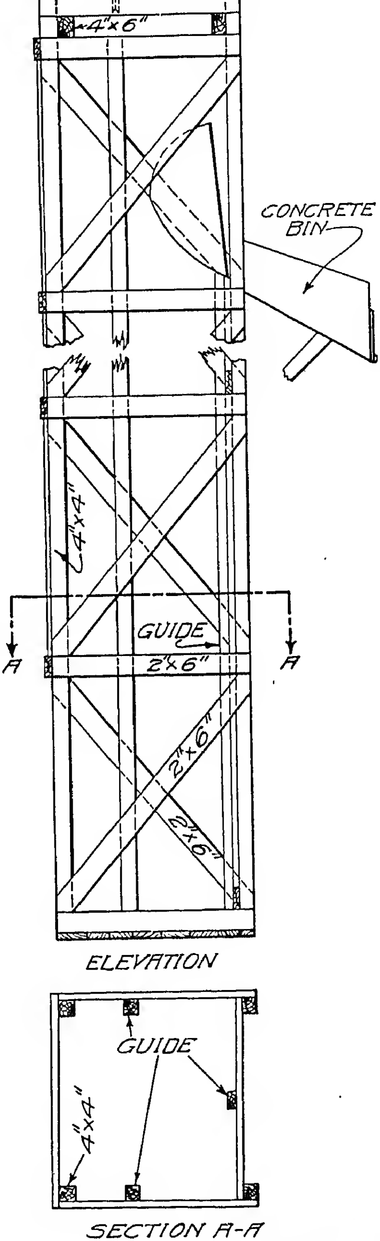


Fig. 139. Details of Hoisting Tower with Automatic Dumping Bucket

former size being generally used, though with good runways, a man can handle 4 cubic feet of stone or sand in a well-constructed wheelbarrow.

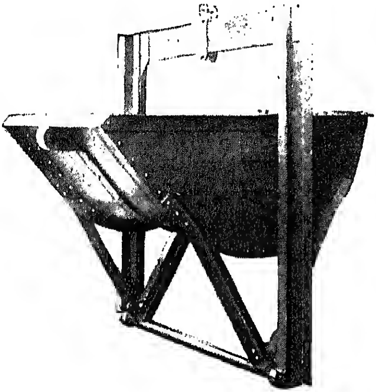


Fig. 140. Ransome Hoist Buckets for Concrete
Courtesy of Ransome Concrete Machinery Company, Chicago

In ordinary massive concrete construction, as foundations, piers, etc., where it is not necessary to hoist the concrete after it is mixed, the mixer is usually elevated so that the concrete can be discharged directly into wheelbarrows, carts, cars, or a chute from which the wheelbarrows or carts are filled. It is much better to discharge the concrete into a receiving chute than to discharge it directly into the conveyor. The chute can be emptied while the mixer is being charged and rotated; while, if the concrete is discharged directly into wheelbarrows or carts, there must be enough

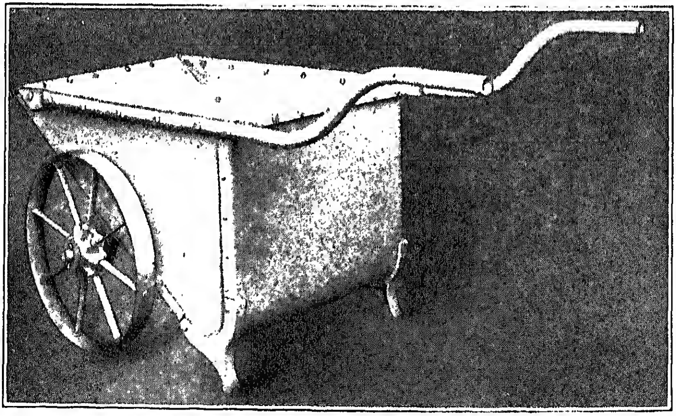


Fig. 141. Typical Concrete Barrow
Courtesy of Ransome Concrete Machinery Company, Chicago

wheelbarrows or carts *waiting* to receive the discharge, or the man charging the mixer will be idle while the mixer is being discharged. A greater objection is, that if the man in charge of the mixer finds

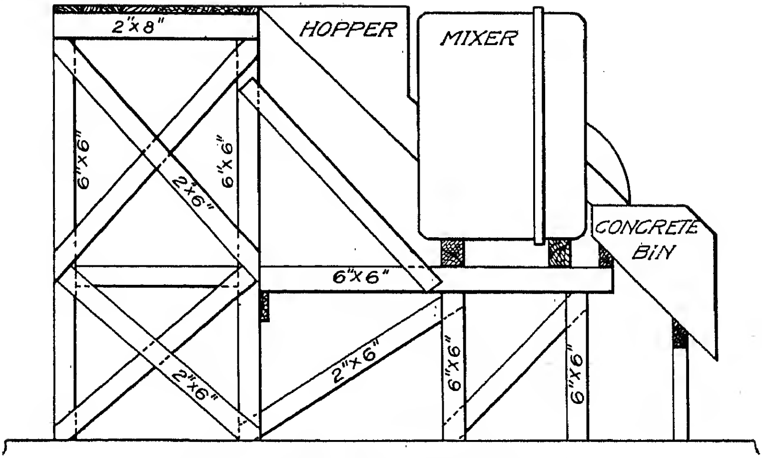


Fig. 142. Details of Concrete Mixer Erected

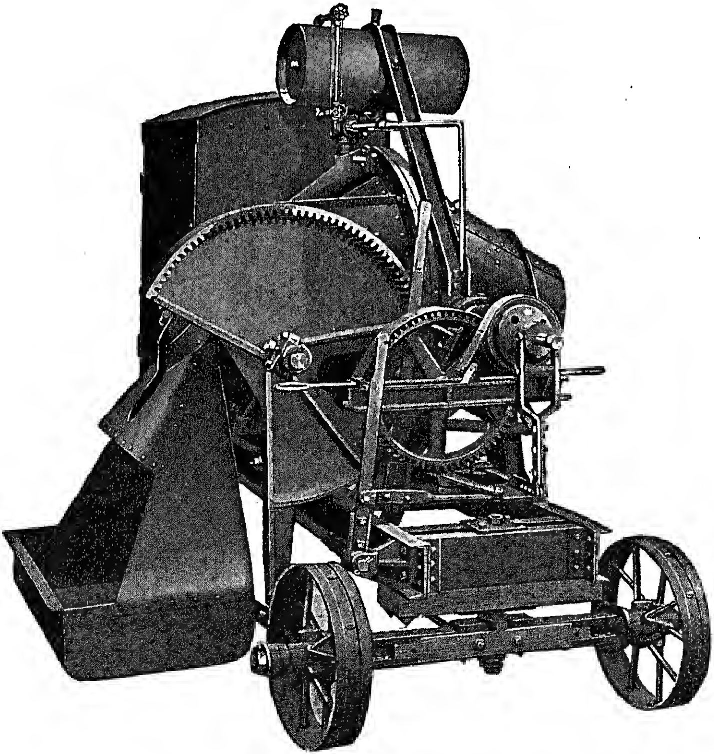


Fig. 143. Smith Concrete Mixer on Truck with Gasoline Engine, Power

that the charging men or conveying men are waiting, he is very apt to discharge the concrete before it is thoroughly mixed, in an effort to keep all the men busy. A platform is built at the elevation of the top of the hopper, through which the materials are fed to the mixer, Fig. 142. This is a rather expensive operation for mixing concrete, and should always be avoided when possible.

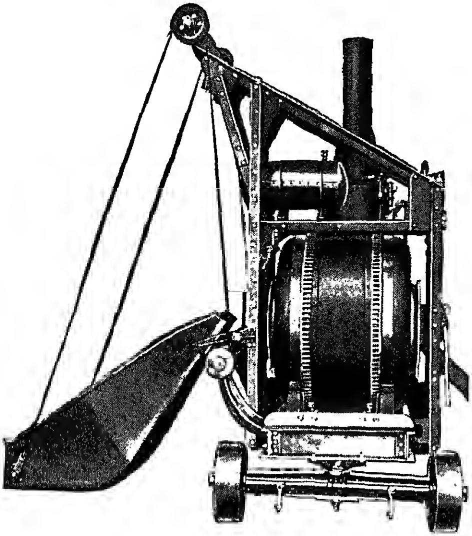


Fig. 144. Koehring Steam-Driven Concrete Mixer with Side Loader and Water Measuring Tank
Courtesy of Koehring Machine Company, Milwaukee, Wisconsin

In Fig. 143 is shown a charging elevator manufactured by the T. L. Smith Company of Milwaukee, Wisconsin. The bucket is raised and lowered by the same engine by which the concrete is mixed, and it is operated by the same man. The capacity of the charging bucket is the same as that of the mixer.

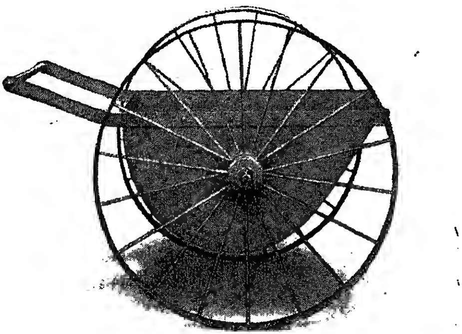


Fig. 145. Typical Concrete Cart
Courtesy of Ransome Concrete Machinery Company, Chicago

In Fig. 144 is shown an automatic loading bucket which has been devised by the Koehring Machine Company for charging the mixers made by them. The bucket is operated by a friction clutch, and is provided with an automatic stop.

In using either make of these charging buckets, it is necessary to use

Transporting Mixed Concrete. Concrete is usually transported by wheelbarrows, carts, cars, or derricks, although other means are frequently used. It is essential, in handling or transporting concrete, that care be taken to prevent the separation of the stone from the mortar. With a wet mixture, there is not so much danger of the stone separating. Owing to the difference in the time of setting of Portland cement and natural cement, the former can be conveyed much farther and with less danger of the initial setting taking place before the concrete is deposited. When concrete is mixed by hand, wheelbarrows are generally used to transport the concrete; and they are very often used, also, for transporting machine-mixed concrete. The wheelbarrows used are of the type shown in Fig. 141. About $1\frac{1}{2}$ cubic feet of wet concrete is the average load for a man to handle in a wheelbarrow.

Fig. 145 shows a cart of the Ransome make, for transporting concrete. The capacity of these carts is 6 cubic feet, and one man can push or pull them over a plank runway. The runways are made of two planks, and in width are at least a foot wider than the distance between the wheels. These planks are fastened together on the back with 2- by 6-inch cross pieces, and made in sections so that they can be handled by four men.

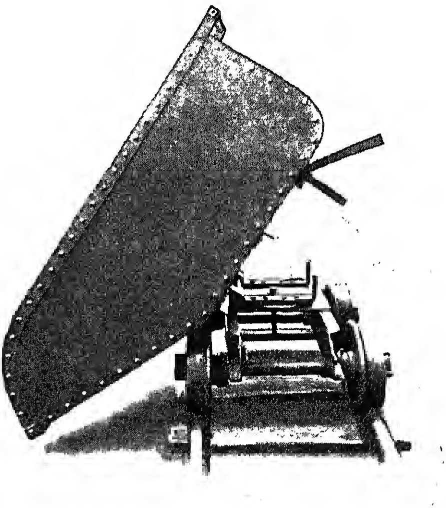


Fig. 146. Typical Rotary Dump Car
Courtesy of Ransome Concrete Machinery Company, Chicago

When it is necessary to convey concrete a longer distance than is economical by means of wheelbarrows or carts, a dumping car, run on

bination of car and derrick work is easily made by using flat cars with derrick buckets.

Boilers. Upright tubular boilers are generally used to supply steam for concrete mixers and hoists operated by steam engines, when they are isolated. For the smaller sizes of mixers, the boilers are on the same frame as the engine and mixer. Fig. 131 shows a Smith-Chicago mixer, engine, and boiler mounted on the same frame. In a similar manner the boiler is often fastened to the same frame as the hoisting engine. This arrangement cannot be used for the larger sizes of mixers and hoists, as they are too heavy to be handled conveniently.

When it is possible, the mixer and hoists should be supplied with steam from one centrally located boiler. A portable boiler is then generally used.

SPECIFICATIONS FOR CONSTRUCTION PLANTS

Woodworking Plant. A portable woodworking plant can very often be used to advantage in shaping the lumber for the forms, when a large building is to be erected. The plant can be set near the site of the building to be erected, and the woodworking done there. The machinery for such a plant should consist of a planer adapted for surfacing lumber on three sides, a ripsaw, and a crosscut circular saw; in some cases, a band saw can be used to advantage. Usually, the difference in cost between surfaced and unsurfaced lumber is so slight that the lumber could not be surfaced in a plant of this kind, for the difference in cost; but perhaps it would be more uniform in thickness. In such a plant the ripsaw and the crosscut saw would be found to be the most useful; and, if reasonable care is taken, this machinery will soon pay for itself. It is often difficult to get work done at a planing mill when it is wanted; and if a contractor has his own woodworking machinery, he will be independent of any planing mill. A plant of this kind can be operated by a steam or gasoline engine or by an electric motor.

Plant for Ten-Story Building. The plant used by Cramp and Company in constructing a reinforced-concrete building in Philadelphia will be described to show the arrangement of the plant rather

forced concrete, except that the interior columns in the lower floors were constructed of angles and plates and fireproofed with concrete. The power plant for the building is located at a level of about seven feet below the basement floor. The hoisting shaft is built in the elevator shaft located in the rear of the building. The hoisting tower

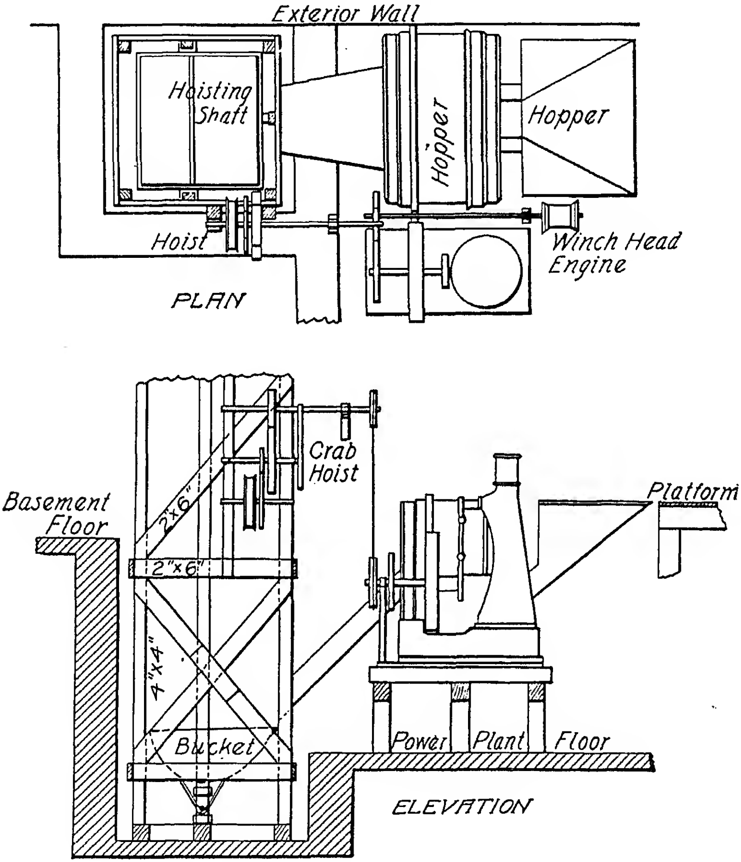


Fig. 147. Concrete Plant for Ten-Story Building

is constructed of four 4- by 4-inch corner-posts, and well braced with 2- by 6-inch plank. Two guides are placed on opposite sides; also one on the front, Fig. 147. The front guide was made in lengths equal to the height of different floors of the building. Fig. 147 shows the location of the machinery, all of which is of the Ransome make. The

out, and the concrete was dumped automatically, by the bucket tipping forward, the bucket righting itself as soon as it began to descend.

The capacity of the mixer and hoisting bucket, per batch, was 20 cubic feet. A 9- by 9-inch, 20-horsepower vertical engine was used to mix and hoist the concrete, steel, structural steel for columns, and lumber for the forms. A 30-horsepower boiler was used to supply the steam; this boiler was located several feet from the engine, and is not shown in the plan view of the plant. A Ransome friction crab hoist was used to hoist the concrete, and was connected to the engine by a sprocket wheel and chain. The steel and lumber were hoisted by means of a rope, wrapped three or four times around a winch head which was on the same shaft as the mixer. The rope extended vertically up from the pulley, through a small hole in each floor, to a small pulley at the height required to hoist the lumber or steel; it then extended horizontally to another pulley at the place where the material was to be hoisted, the rope descending over the pulley to the ground. A man was stationed at the engine to operate the rope. There were two rope haulages operated from the pulley on the engine shaft, one being used at a time. On being given the signal, the operator wrapped the rope around the winch head three or four times, kept it in place, and took care of the rope that ran off the pulley as material was being hoisted.

Wheelbarrows were used in charging the mixer, and handcarts were used in distributing the concrete. The runways were made by securely fastening two 2- by 10-inch planks together in sections of 12 to 16 feet, which were handled by two men. By keeping the runway in good condition, two men were usually able to distribute the concrete, except on the lower floors, and when it was to be transported the full length of the building. The capacity of the carts was 6 cubic feet each. Concrete for the ninth floor was hoisted and placed at the rate of 15 cubic yards per hour.

Plant for the Locust Realty Company Building. The plant used for constructing a five-story reinforced-concrete building, 117 feet by 200 feet, for the Locust Realty Company, by Moore and Company, is a good example of a centrally located plant. Near the center of the building is an elevator shaft, in which was constructed the framework for hoisting the concrete. Fig. 148 shows the arrange-

necessary to wheel the materials up an incline. An excavation was made below the level of the basement floor for the hoisting bucket. The mixing was done by a steam engine located on the same frame as the mixer. The concrete was hoisted by a hoisting engine which was located about twenty feet from the shaft. A small hoisting engine was also used for hoisting the steel and lumber used for forms; as this engine was located some distance from the rest of the plant, it is not shown in Fig. 148. The three engines are supplied with steam from a portable boiler which is located as shown in the figure. The efficiency of this plant was shown in the mixing and hoisting of the concrete for the second floor, when 240 cubic yards were mixed and hoisted in 16 hours, or at an average rate of 1 cubic yard in 4 minutes.

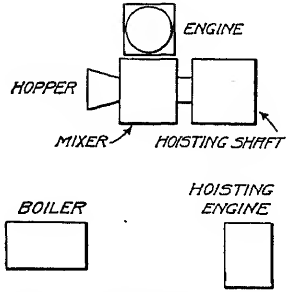


Fig. 148. Diagrammatic Layout for Typical Concrete Plant

All materials were delivered at the front of the building; it was necessary, therefore, to transport the cement, sand, and stone about 100 feet to the mixer. This was done by means of wheelbarrows of 4 cubic feet capacity, which were especially designed and made for the Moore Company. A 1:2:4 concrete was used, mixed in batches of 14 cubic feet. The materials for a batch, therefore, consisted of 2 bags of cement, 1 wheelbarrow of sand, and 2 wheelbarrows of stone.

The lumber for the forms was $1\frac{1}{4}$-inch plank, except the support and braces. Details of the forms will be given and discussed under the heading of "Forms".

Concrete Plant for Street Work. A self-propelling mixing and spreading machine has been found very desirable for laying concrete base for street pavements. Fig. 149 illustrates a plant of this kind, devised by the Municipal Engineering and Contracting Company.

The mixer is of the improved cube type, mounted on a heavy truck frame. The concrete is discharged into a specially designed bucket, which receives the whole batch and travels to the rear on a

truck which is about 25 feet long. The head of the truck is supported by guys, and also by a pair of small wheels near the middle of the

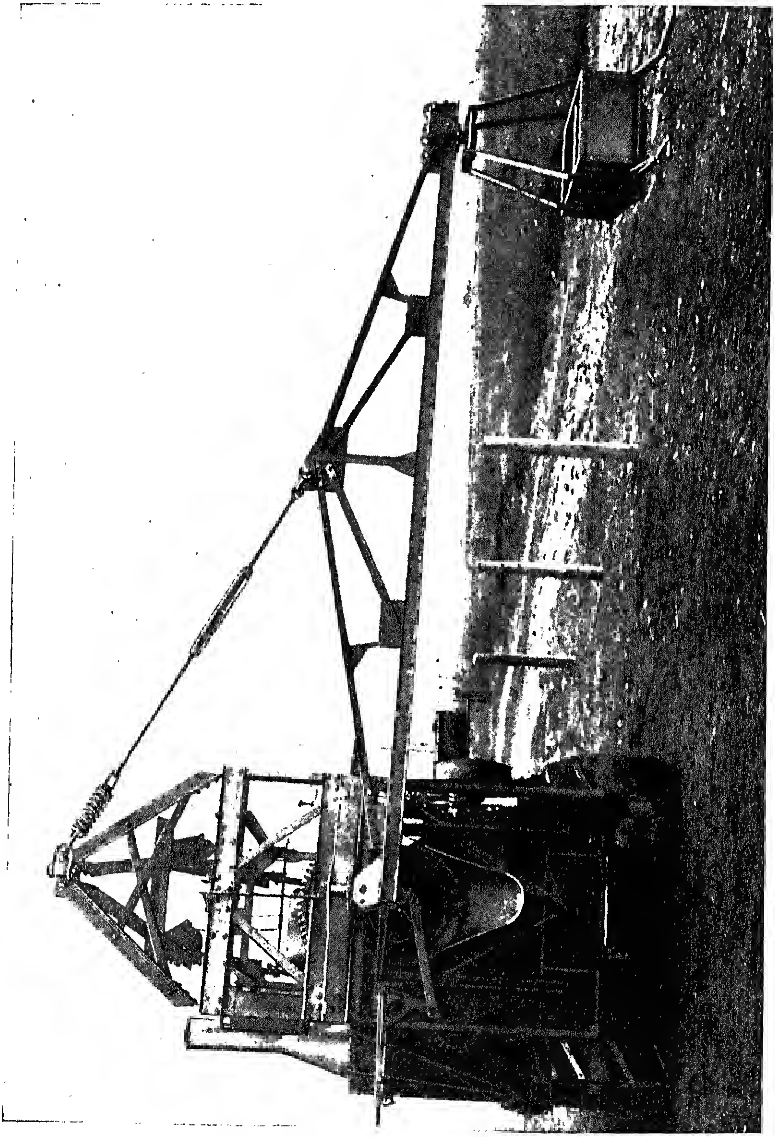


Fig. 149. Rear View of Special Street Mixer Showing Swinging Concrete Depositing Boom
Courtesy of Municipal Engineering and Contracting Company, Chicago

truck, which rest on the graded surface of the street. The truck or

horizontal swing of about 170 degrees, so that a street 50 feet wide is covered. An inclined track is also constructed, on which a bucket is operated for elevating and charging the mixer. The bucket is loaded while resting on the ground, with the proper ingredients for a batch, from the materials that have been distributed in piles along the street. The bucket is then pulled up the incline, and the contents dumped into the mixer. An automatic water-measuring supply tank, mounted on the upper part of the frame, insures a uniform amount of water for each batch mixed. The power for hoisting, mixing, and distributing the concrete, and propelling the machine was furnished by a 16-horsepower gasoline engine of the automobile type. The machine can be moved backward as well as forward, and is supplied with complete steering gear.

MISCELLANEOUS OPERATIONS

Concrete-Block Machines. There are two general types of hollow-concrete-block machines on the market—those with a *vertical*

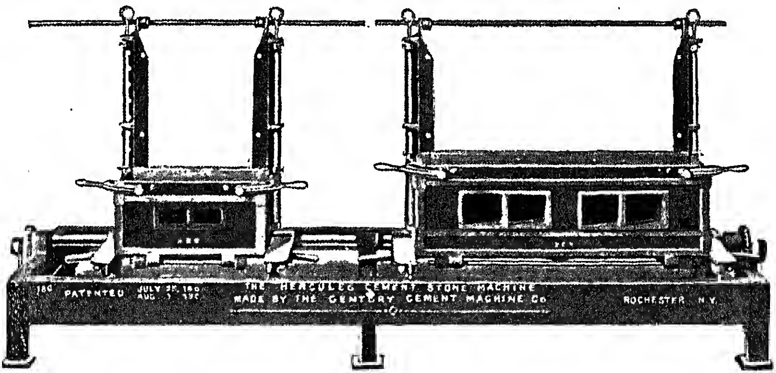


Fig. 150. "Hercules" Cement Stone Machine
Courtesy of Century Cement Company, Rochester, New York

face and those with a *horizontal* face. In making blocks with the vertical-faced machine, the face of the block is in a vertical position when molded, and is simply lifted from the machine on its base plate. The horizontal-faced type of block is made with the face down, the face plate forming the bottom of the mold. The cores are withdrawn horizontally, or the mold is turned over and the core is taken out vertically; the block is then ready for removal. The principal difference in the two types of machine is that, if it is desired

to put a special facing on the block, it is more convenient to do it with a horizontal-faced machine. With the vertical-faced machine, the special facing is put on by the use of a parting plate. When the parting plate is removed, the two mixtures of concrete are bonded together by tamping the coarser material into the facing mixture.

Fig. 150 shows a Hercules machine. The foundation parts can be attached for making any length of block up to 6 feet. The illustration shows two molds of different lengths attached. These machines are constructed of iron and steel, except that the pallets (the plates on

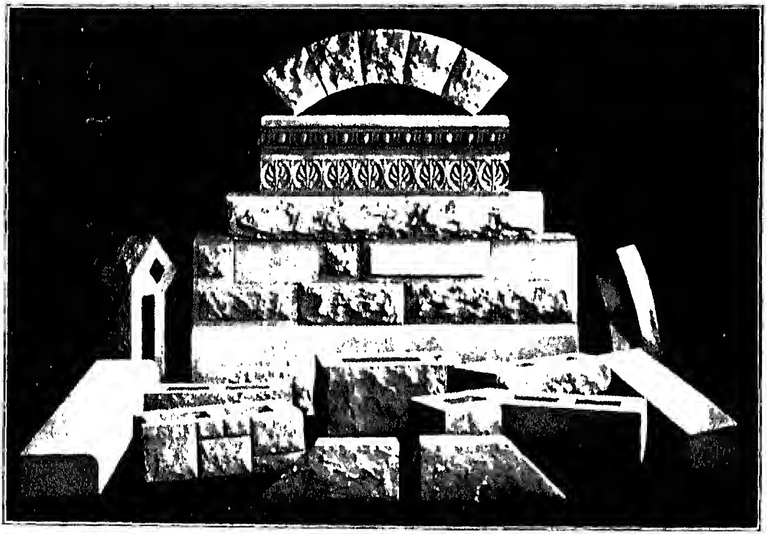


Fig. 151. Group of Blocks made on "Hercules" Machine

which the blocks are taken from the machine) may be either wood or steel. This type of machine is the horizontal or face-down machine.

In Fig. 151 are shown a group of the various forms which may be made. The figure also illustrates the facility with which concrete may be utilized for ornamental as well as structural purposes.

Another machine of the face-down type is shown in Fig. 152. This machine, the Ideal, is simple in construction and operation, and, being portable, it is convenient to operate. In making blocks with this machine, the cores are removed by means of a lever, while the block is in the position in which it was made. The mold and

and gives a better block.

In Fig. 154 is shown a Hobbs face-down, wet-process block machine. The front and sides of the machine can be let down, thus facilitating the removal of the blocks. The cores are shown withdrawn in the figure.

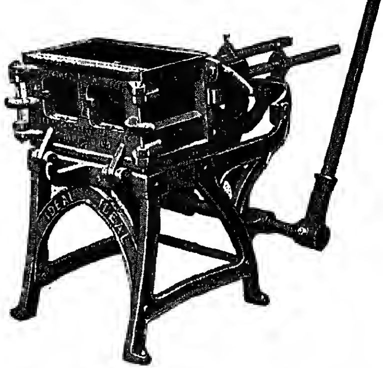


Fig. 152. "Ideal" Concrete Block Machine
Courtesy Ideal Concrete Machinery Company, Cincinnati, Ohio

Cement-Brick Machines. Fig. 155 shows a machine for making cement brick. Ten bricks, $2\frac{3}{8}$ by $3\frac{7}{8}$ by 8 inches, are made at one operation. By using a machine in which the bricks are made on the side, a wetter mixture of concrete can be used than if they are made on the edge. The concrete usually consists of a mixture of 1 part Portland cement and 4 parts sand. The curing of these bricks is the same as that for concrete blocks. In making these bricks, a number of wood pallets are required, as the brick should not be removed from the pallet until the concrete has set.

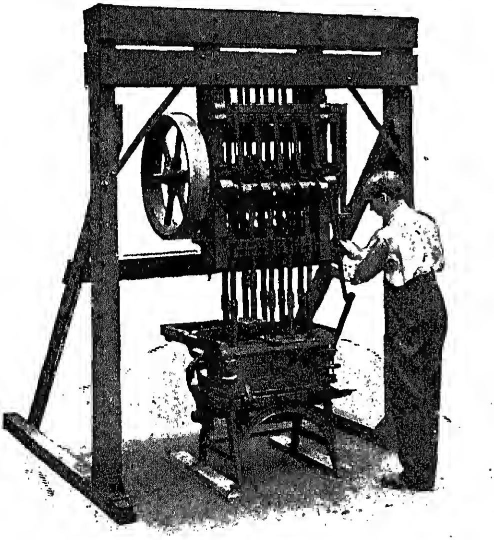


Fig. 153. "Ideal" Automatic Tamper Used in Connection with Block Machine
Courtesy of Ideal Concrete Machinery Company, Cincinnati, Ohio

Sand Washing. It sometimes becomes necessary to wash dirty sand, which can easily be obtained, while clean

sand can be secured only at a high cost. If only a small quantity is to be washed, it may be done with a hose. A trough should be built about 8 feet wide and 15 feet long, the bottom having a slope of about 19 inches in its entire length. The sides should be about 8 inches high at the lower end, and increase gradually to a height of about 36 inches at the upper end. In the lower end of the trough should be a gate about 6 inches high, sliding in guides so that it can be easily removed. The sand is placed in the upper end of the trough, and a stream of water is played on it. The sand and water flow down the trough, and the dirt passes over the gate with the overflow water. With a trough of the above dimensions, and a stream of water from a $\frac{3}{4}$-inch hose, three cubic yards of sand should be washed in an hour.

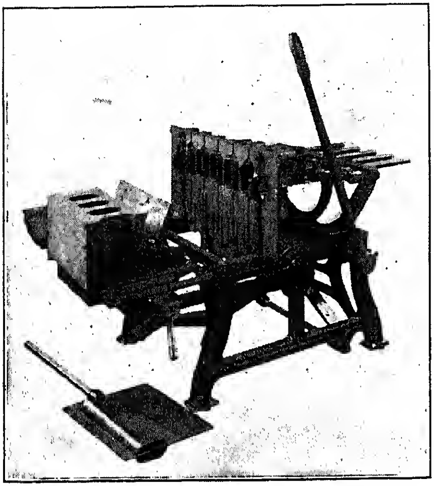


Fig. 154. Hobbs Face-Down, Wet-Process Concrete Block Machine
Courtesy of Hobbs Concrete Machinery Company, Detroit, Michigan

Concrete mixers are often used for washing sand. The sand is dumped into the mixer in the usual manner and the water is turned on. When the mixer is filled with water so that it overflows at the discharge end, the mixer is started. By revolving the mixer, the water is able to separate the dirt from the sand,

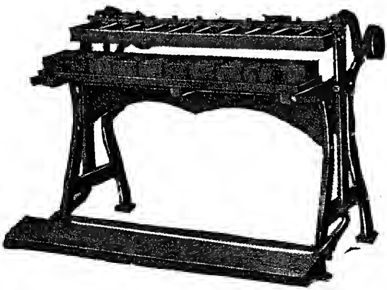


Fig. 155. "Century" Cement Brick Machine

General Requirements. In actual construction work, the cost of forms is a large item of expense and offers the best field for the exercise of ingenuity. For economical work, the design should consist of a repetition of identical units; and the forms should be so devised that it will require a minimum of nailing to hold them, and of labor to make and handle them. Forms are constructed of the cheaper grades of lumber. To secure a smooth surface, the planks are planed on the side on which the concrete will be placed. Green lumber is preferable to dry, as it is less affected by wet concrete. If the surface of the planks that is placed next to the concrete is well oiled, the planks can be taken down much easier, and, if kept from the sun, they can be used several times.

Crude oil is an excellent and cheap material for greasing forms, and can be applied with a whitewash brush. The oil should be applied every time the forms are used. The object is to fill the pores of the wood, rather than to cover it with a film of grease. Thin soft soap, or a paste made from soap and water, is also sometimes used.

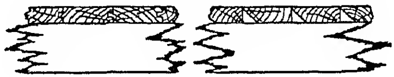


Fig. 156. Typical Form of Construction Showing Tongued-and-Grooved and Beveled-Edge Boards

In constructing a factory building of two or three stories, usually the same set of forms is used for the different floors; but when the building is more than four stories high, two or more sets of forms are specified, so as always to have one set of forms ready to move.

The forms should be so tight as to prevent the water and thin mortar from running through and thus carrying off the cement. This is accomplished by means of tongued-and-grooved or beveled-edge boards, Fig. 156; but it is often possible to use square lumber if it is wet thoroughly, so as to swell it before the concrete is placed. The beveled-edge boards are often preferred to tongued-and-grooved boards, as the edges tend to crush as the boards swell, and beveling prevents buckling.

Lumber for forms may be made of 1-inch, $1\frac{1}{2}$-inch, or 2-inch plank. The spacing of studs depends in part upon the thickness of concrete to be supported, and upon the thickness of the boards on which the concrete is placed. The size of the studding depends upon

the height of the wall and the amount of bracing used. Except in very heavy or high walls, 2- by 4-inch or 2- by 6-inch studs are used. For ordinary floors with 1-inch plank, the supports should be placed about 2 feet apart; with $1\frac{1}{2}$-inch plank, about 3 feet apart; and with 2-inch plank, 4 feet apart.

The length of time required for concrete to set depends upon the weather, the consistency of the concrete, and the strain which is to come on it. In good drying weather, and for very light work, it is often possible to remove the forms in 12 to 24 hours after placing the concrete, if there is no load placed on it. The setting of concrete is greatly retarded by cold or wet weather. Forms for concrete arches and beams must be left in place longer than in wall work, because of the tendency to fail by rupture across the arch or beam. In small, circular arches, like sewers, the forms may be removed in 18 to 24 hours, if the concrete is mixed dry; but if wet concrete is used, in 24 to 48 hours. Forms for large arch culverts and arch bridges are seldom taken down in less than 28 days. The minimum time for the removal of forms should be:

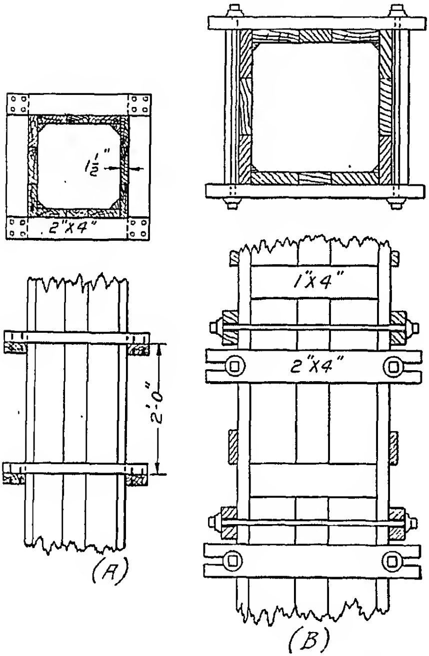


Fig. 157. Forms for Columns, (a) Common Method of Construction; (b) Method in Constructing Harvard Stadium

For bottom of slabs and sides of beams and girders, 7 days
For bottom of beams and girders, 14 days
For columns, 4 days
For walls, not loaded, 1 to 2 days
For bridge arches, 28 days

The concrete should be thoroughly examined before any forms are removed. Forms must be taken down in such a way as not to deface the structure or to disturb the remaining supports.

Forms for Columns. In constructing columns, the forms are usually erected complete, the full height of the columns, and concrete is dumped in at the top. The concrete must be mixed very wet, as it cannot be rammed very thoroughly at the bottom, and care must be taken not to displace the steel. Sometimes the forms are constructed in short sections, and the concrete is placed and rammed as the forms are built. The ends of the bottom of the forms for the girders and beams are usually supported by the column forms. To give a beveled edge to the corner of the columns, a triangular strip is fastened in the corner of the forms.

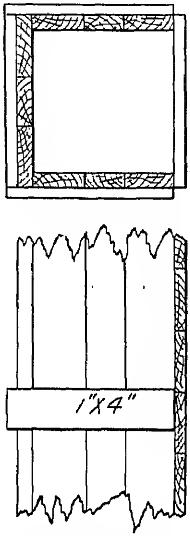


Fig. 158. Forms for Square Columns

Fig. 157-*A* shows the common way, or some modification of it, of constructing forms for columns. The plank may be 1 inch, 1½ inches, or 2 inches thick; and the cleats are usually 1 by 4 inches and 2 by 4 inches. The spacing of the cleats depends on the size of the columns and the thickness of the vertical plank.

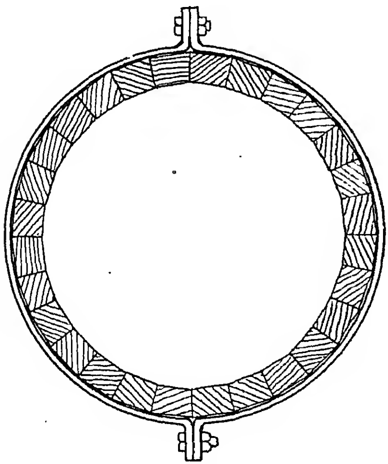


Fig. 159. Forms for Round Columns

Fig. 157-*B* shows column forms similar to those used in constructing the Harvard stadium. The planks forming each side of the column are fastened together by cleats, and then the four sides are fastened together by slotted cleats and steel tie-rods. These forms can be quickly and easily removed.

Fig. 158 shows a column form in which concrete is placed and rammed as the form is constructed. Three sides are erected to the full height, and the steel

buildings. Fig. 159 shows a form that has been used for this type of column. The columns for which these forms were used were 20 inches in diameter, and had a star-shaped core made of structural steel. The forms for each column were made in two parts and bolted together. The sides were made of 2- by 3-inch plank surfaced on all four sides, beveled on two, and held in place by steel bands, ¼ by 2½

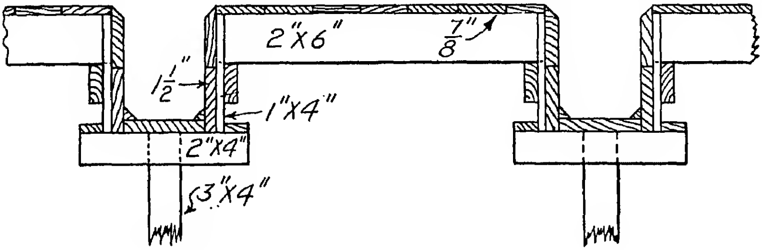


Fig. 160. Forms for Beams and Slabs

inches, spaced about 2½ feet apart. One screw in the outer plank of both parts of each band, together with a few intermediate screws, held the planks in place. The building for which these forms were made was ten stories in height. Enough forms were provided for two stories, which was sufficient, as they could be removed when the concrete had been in place one week. Later, these same forms were used in constructing the interior columns of a six-story building. Some difficulty was experienced in removing these forms, owing to

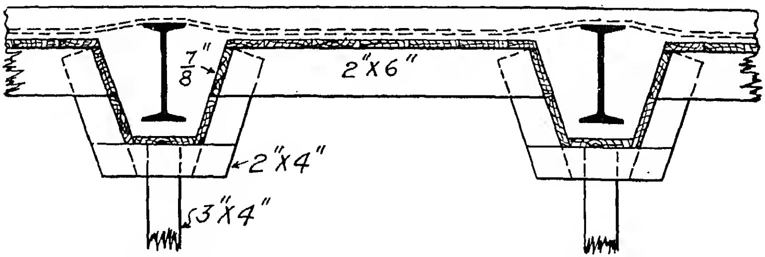


Fig. 161. Forms for Reinforced Concrete Slab Supported by I-Beams

the concrete sticking to the plank. But had the forms been made in four sections, instead of two, and well oiled, it is thought that this trouble would have been avoided. Columns constructed with forms as shown in Fig. 159 will not have a round surface, but will consist of many flat surfaces, 2½ inches wide. If a perfectly round column

the concrete to the desired radius. Forms for octagonal columns can be made in a manner somewhat similar to these just described.

Forms for Beams and Slabs. A very common style of form for beam and slab construction is shown in Fig. 160. The size of the different members of the forms depends upon the size of the beams, the thickness of the slabs, and the relative spacing of some of the members. If the beam is 10 by 20 inches, and the slab is 4 inches thick, then 1-inch plank supported by 2- by 6-inch timbers spaced 2 feet apart will support the slab. The sides and bottom of the beams are enclosed by $1\frac{1}{2}$-inch or 2-inch plank supported by 3- by 4-inch posts spaced 4 feet apart.

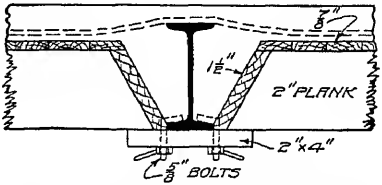


Fig. 162. Forms for Reinforced Concrete Slab between I-Beams

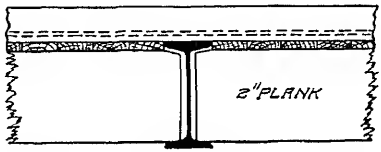


Fig. 163. Forms for Floor-Slab on I-Beams

In Fig. 161 are shown the forms for a reinforced-concrete slab, with I-beam construction. These forms are constructed similarly to those just described.

A slab construction supported on I-beams, the bottom of which is not covered with concrete, may have forms constructed as shown in Fig. 162. This method of constructing forms was designed by Mr. William F. Kearns (Taylor and Thompson, "Plain and Reinforced Concrete").

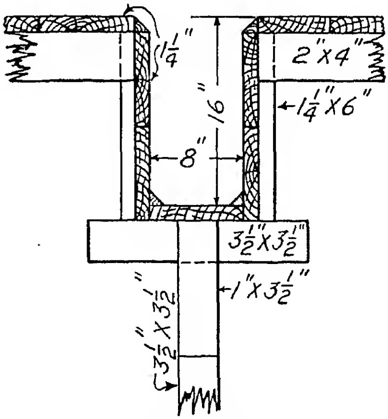


Fig. 164. Beam and Slab Forms for Locust Realty Company Building

The construction of forms for a slab that is supported on the top of I-beams is a comparatively simple process, as shown in Fig. 163. In any form of I-beam and slab construction, the forms can be constructed to carry the combined weight of the concrete

concrete, it is not so easily done as when the haunch rests on the bottom flange, as shown in Fig. 162, or when it is a flat plate, as shown in Fig. 163.

Forms for Locust Realty Company Building. The forms used in constructing the building for the Locust Realty Company (the mixing plant has already been described), present one rather unusual feature. The lumber for the slabs, beams, girders, and columns was all the same thickness, $1\frac{1}{4}$ inches. Fig. 164 shows the details of the forms for the beams and slabs. The beams are spaced about 6 feet apart, and are 8 by 16 inches; the slab is 4 inches thick. A notch is cut into the $1\frac{1}{4}$- by 6-inch strip on the side of the beams, to support the 2- by 4-inch strip under the plank which supports the concrete for the slab. The posts supporting the forms are $3\frac{1}{2}$- by $3\frac{1}{2}$-inch, and are braced by two 1- by 6-inch boards which are spaced about 3 feet apart and extend in the direction of the beams.

Fig. 165 shows the forms for the columns. The planks for each side of the column are held together by the 1- by 4-inch strip, and, when erected in place, are clamped by the 2- by 4-inch strip. A large opening is always left at the bottom of the form for each column, so that all shavings and sawdust can be removed. This opening is closed just before the concrete is deposited.

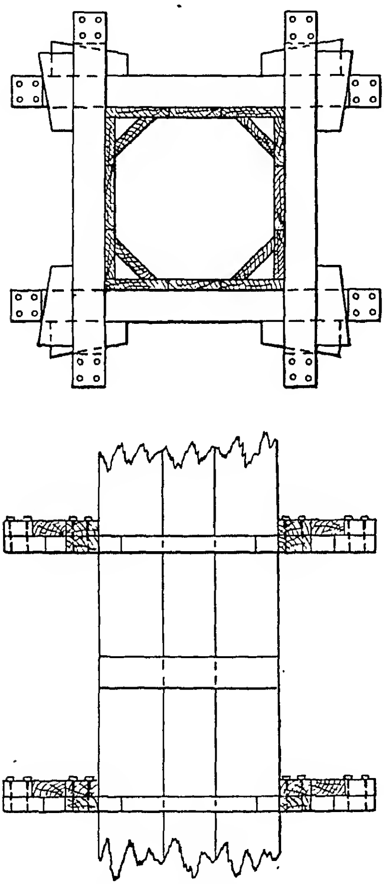


Fig. 165. Column Forms for Locust Realty Company Building

Cost of Forms for Buildings. An analysis of the cost of forms

neering. The basis of his estimate is made on using $\frac{7}{8}$-inch by 6-inch tongued-and-grooved lumber for slab forms; $1\frac{3}{4}$-inch dressed plank for the sides and bottom of the beams and girders; and posts 4- by 4-inch, spaced 6 feet center to center. He makes the further assumption that it cost \$20.00 per thousand feet of lumber to make and set one floor of forms; that it cost \$15.00 per thousand feet of lumber to strip the forms and reset them on the next floor; and that it cost about \$8.00 per thousand feet to strip the forms and lower them to the ground.

With the size of the beams and girders as shown in Fig. 166, Mr. Lamb states that it will take an average of 4 feet, board measure, to erect each square foot of floor area. The basis of his estimate is as follows: That 1.5 board feet of lumber per square foot of floor is required for the slab; that for every square foot of beam surface, including the bottom, 3.2 board feet per square foot is required; and that for each square foot of girder, including the bottom, 3.6 board feet of lumber is required. Taking these figures, for the panel shown, the slab will require 1.5 board feet per square foot; the beams, which are 8- by 18-inch, will have 3 feet 8 inches of surface per lineal foot; and multiplying this by 3.2 board feet per square foot, and dividing by 7.5 feet, the distance center to center of beams, we find that 1.56 board feet per square foot of floor surface is required. Taking the girder in the same way, with 4 feet 8 inches of surface, multiplied by 3.6 board feet, and divided by 18 feet, the distance center to center of girders, we find that .94 board foot per square foot of floor is required. The total of the lumber required, then, is 1.5 board feet for the slab, 1.56 board feet for the beam, and .94 board foot for the girders—a total of 4 board feet per square foot of floor area.

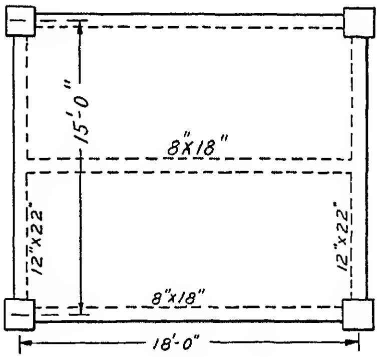


Fig. 166. Diagram of Forms

In this estimate for an eight-story building, three sets of forms

Roof.	Stripping the sixth floor, resetting, altering to form valleys, and finally stripping roof and lowering forms to ground, 4 board feet at 2.6 cents		$0.104
Eighth Floor.	Stripping the fifth floor, resetting, and finally stripping and lowering forms to ground, 4 board feet at 2.3 cents		.092
Seventh Floor.	Stripping the fourth floor, resetting, and finally stripping and lowering forms to ground, 4 board feet at 2.3 cents		.092
Sixth Floor.	Cost, same as for the fourth floor		.060
Fifth Floor.	Cost, same as for the fourth floor		.060
Fourth Floor.	Stripping the first floor, and resetting, 4 board feet at 1.5 cents		.060
Third Floor.	Cost, the same as for the first floor		.184
Second Floor.	Cost, same as for the first floor		.184
First Floor.	Making and setting forms, 4 board feet at 2 cents	$0.080	
	Material, 4 board feet at 2.6 cents	.104	.184
			9)1.020
	Average cost per square foot of surface		$0.113

To this average cost of 11.3 cents, 10 per cent should be added for waste, breakage, nails, etc.; and if two sets of forms are used, the third floor would cost 6 cents per square foot, and the seventh floor 6 cents, giving an average of 9.6 cents per square foot.

In estimating the cost of the forms for the columns, it is assumed that making and placing the forms for the basement columns will cost about $26.00 per thousand; the cost of stripping and resetting, $16.00 per thousand; and 3.1 square feet of lumber is required for each square foot of column surface.

Eighth Story.	Stripping sixth story, resetting and altering, finally stripping eighth story and lowering to ground, 3.1 board feet at 2.2 cents		$0.068
Seventh Story.	Stripping fifth story, resetting, and finally stripping and lowering to ground, 3.1 board feet at 1.9 cents		.059
Sixth Story.	Cost, same as second story		.050
Fifth Story.	Cost, same as second story		.050
Fourth Story.	Cost, same as second story		.050
Third. Story.	Cost, same as second story		.050
Second Story.	Stripping basement columns and resetting, 3.1 board feet at 1.6 cents		.050
First Story.	Cost, same as for the basement columns		.162
Basement.	Material, 3.1 board feet at 2.6 cents	$0.081	
	Making and setting, 3.1 board feet at 2.6 cents	.081	
		$0.162	.162
			9)0.701
	Average cost per square foot of surface		$0.077

To this average cost of 7.7 cents per square foot of column surface,

of forms are required, the second-story cost would be 16.2 cents, and the sixth 5.9 cents, giving an average cost per square foot of 9.1 cents.

The student should remember that this lumber has a value after it has been removed from the building, and that this value should be deducted from the total to find the actual cost of the forms.

Cost of Forms for Garage. Some interesting cost data are given by Mr. Reygondeau de Gratresse, Assoc. M. Am. Soc. C. E. in *Engineering-Contracting,* on the cost of forms used in erecting a reinforced-concrete garage in Philadelphia. The building was 53 feet wide, 200 feet long, and four stories high; also, there was a mezzanine floor. Tongued-and-grooved lumber $\frac{7}{8}$ inch thick was used for the slab forms, and $1\frac{3}{4}$-inch plank for the beams and girders. The area of the 1,740 cubic yards of concrete covered by forms was:

		Sq. Ft.
Footings		4,000
Columns		20,000
Floors and Girders		70,000
	Total	94,000

For this work, 170,000 feet, board measure, of new lumber, and 50,000 feet, board measure, of old lumber was used, the cost being:

50,000 ft. B. M. at $13	$ 650
170,000 ft. B. M. at $26	4,420
220,000 ft. B. M. at $23	$5,070

Since 220,000 feet, board measure, were used for the 1,740 cubic yards, there were 126 feet, board measure, per cubic yard of concrete.

New forms were made for each floor, except the sides of the girders, which were used over for each floor, where the sizes would admit of this being done. The props under the girders were allowed to remain in place throughout the building until the entire job was completed. The forms for the roof were made entirely of the material used on the floors below. The area of concrete covered by the new lumber was approximately 80,000 square feet. This gives a cost for lumber of 6.4 cents per square foot.

A force of fifteen carpenters, working under one foreman, framed, erected, and tore down all forms. All the lumber for the carpenters was handled by the laborers excepting when they were at work mixing and placing concrete. The foreman was paid $35 per week, while the carpenters were paid an average of $4.40 for an 8-hour day.

over them was a foreman who received the same wages as the boss carpenter. The forms for a floor were erected in from 8 to 10 days. For the framing, erecting, and tearing down of the forms, the labor cost was about $3,480, which gives a cost of $2 per cubic yard. For the carrying and handling of the lumber, the cost was about $1,914, which gives a cost of $1.10 per cubic yard. This gives a total cost per cubic yard of forms as follows:

	Per Cu. Yd.
Lumber, 126 ft. B. M.	$2.90
Framing, erecting, and tearing down	2.00
Handling lumber	1.10
Total	$6.00

This cost is high, owing to the fact that so little of the lumber was used a second time, there being only from 16 to 20 per cent so

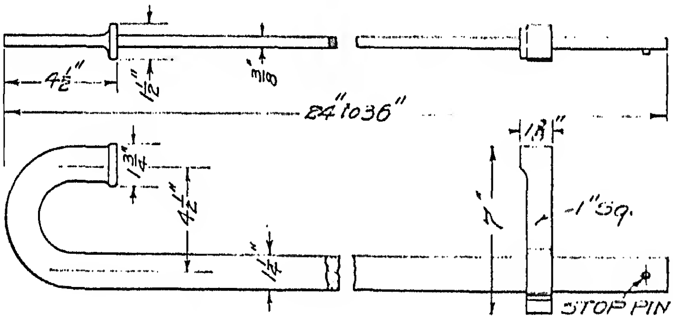


Fig. 167. Typical Adjustable Clamp

used. For the 220,000 feet, board measure, of lumber used on the job, the average cost per thousand for the forms was

	Per M.
Lumber	$23.00
Framing, erecting, and tearing down	15.67
Handling lumber	8.70
Total	$47.37

The cost per square foot of concrete for the area covered was

Lumber	$0.064
Labor	.057
Total	$0.121

The cost per cubic yard for lumber and labor was

Lumber	$2.90
Labor on forms	3.10
Total	$6.00

forming the sides of a beam or girder in place, and also in clamping opposite sides of columns. It is forged from a $1\frac{1}{4}$-inch by $\frac{3}{8}$-inch steel bar, and is held in place by the slotted forging, 1 inch square.

FORMS FOR SEWERS AND WALLS

Forms for Conduits and Sewers. Forms for conduits and sewers must be strong enough not to give way, or to become deformed, while the concrete is being placed and rammed; and must be rigid enough not to warp from being alternately wet and dry. They must be constructed so that they can readily be put up and taken down, and can be used several times on the same job. The forms must give a smooth finish to the interior of the sewer. This has usually been done by covering the forms with light-weight sheet iron.

These forms are usually built in lengths of 16 feet, with one center at each end, and with three to five—depending on the size of the sewer or conduit—intermediate centers in the lengths of 15 feet. The segmental ribs are bolted together. The plank for these forms are made of 2- by 4-inch material, surfaced on the outer side, with the edge beveled to the radius of the conduit. The segmental ribs are bolted together, and are held in place by wood ties 2 by 4 inches or 2 by 6 inches.

Forms of Torresdale Filters. In constructing the Torresdale filters for supplying Philadelphia with water, several large sewers and conduits were built of concrete and reinforced with expanded metal. In section, the sewers were round and the conduits were horseshoe-shaped, with a comparatively flat bottom. The sewers were 6 feet and 8 feet 6 inches, respectively, in diameter, and the forms were constructed similarly to the forms shown in Fig. 168, except that at the bottom the lower side ribs were connected to the bottom rib by a horizontal joint, and the spacing of the ribs was 2 feet 6 inches, center to center. Fig. 169 shows the form for the 7-foot 6-inch conduit. The centering for the 9-foot and 10-foot conduits was constructed similarly to the 7-foot 6-inch conduit, except that the ribs were divided into 7 parts instead of 5 parts as shown in Fig. 169. The spacing of the braces depended on the thickness of the lagging. For lagging 1 inch by $2\frac{1}{2}$ inches, the braces were spaced 18 inches, center

to center; and for 2- by 3-inch lagging, the spacing of the bracing was 2 feet 6 inches.

These forms were constructed in lengths of 8 feet. The lagging

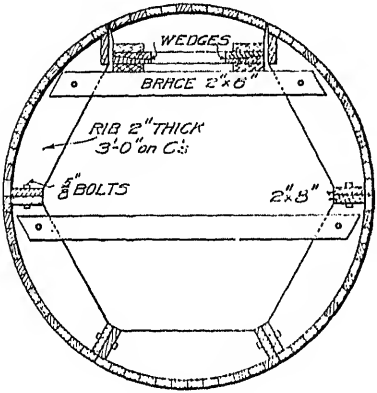


Fig. 168. Center for Round Sewer

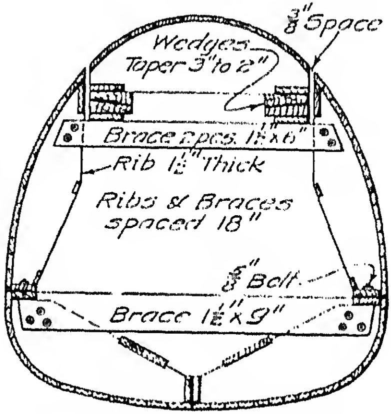


Fig. 169. Form for Construction of Horseshoe-Shaped Conduit

for the smaller sizes of the conduits was 1 inch by $2\frac{1}{2}$ inches, and for the larger sizes 2 by 3 inches; all of this was made of dressed lumber and covered with No. 27 galvanized sheet iron. The bracing

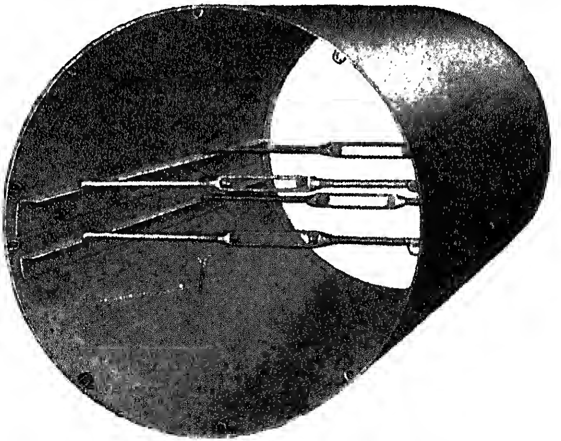


Fig. 170.

required that the centering be left in place for at least 60 hours after the concrete had been placed. It was also required that this work should be constructed in monolithic sections—that is, the contractor could build as long a section as he could finish in a day—and that the sections should be securely keyed together.

Blaw Steel Forms. The Blaw collapsible steel forms, Fig. 170, appear to be the only successful steel forms, so far, in general use. There have been many attempts to devise steel centering for column, girder, and slab construction, but no available system has yet been invented. The main trouble with those used is their liability to leak, tendency to rust, and susceptibility to injury by dents in removing.

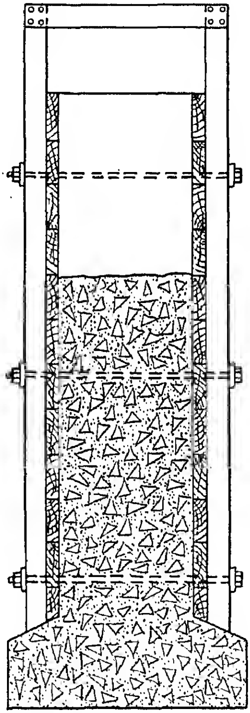


Fig. 171. Typical Forms for Wall

The Blaw collapsible steel centering is in general use for sewer and conduit construction. This centering consists of one or more steel plates about $\frac{1}{8}$ inch thick and bent to the shape required by the interior of the sewer to be constructed. The steel plates are held in shape by angle irons. When set in position, the sections are held rigid by means of turnbuckles, which also facilitate the collapsing of the sections. The adjacent sections are held together by staples and wedges, the former being riveted to the plates as seen in Fig. 170. The sections are usually made five feet long, and in any desired shape or size required for sewer or conduit work. When these forms are used to construct concrete sewers or conduits, the surface of the forms must be well coated with grease or soap, to prevent the concrete from adhering to the steel.

Forms for Walls. The forms for concrete walls should be built strong enough to make sure they will retain their correct position while the concrete is being placed and rammed. In high, thin walls, a great deal of care is required to keep the forms in place so that the

Fig. 171 shows a very common method of constructing these forms. The plank against which the concrete is placed is seldom less than $1\frac{1}{2}$ inches thick, and is usually 2 inches thick. One-inch plank is sometimes used for very thin walls; but even then, the supports must be placed close. The planks are generally surfaced on the side against which the concrete is placed. The vertical timbers that hold the plank in place will vary in size from 2 inches by 4 inches to 4 inches by 6 inches, or even larger, depending on the thickness of the wall, spacing of these vertical timbers, etc. The vertical timbers are always placed in pairs, and are usually held in place by means of bolts, except for thin walls, when heavy wire is often used. If the bolts are greased before the concrete is placed, there is ordinarily not much trouble experienced in removing them. Some contractors place the bolts in short pieces of pipe, the diameter of the pipe being about $\frac{1}{8}$ inch greater than that of the bolt, and the length equal to the thickness of the wall. When the bolts are removed the holes are filled with mortar.

FORMS FOR CENTER OF ARCHES

General Specifications. The centers for stone, plain concrete, and reinforced-concrete arches are constructed in a similar manner. A reinforced-concrete arch of the same span and designed for the same loading will not be so heavy as a plain concrete or stone arch, and the centers need not be constructed so strong as for the other types of arches. One essential difference in the centering for stone arches and that for concrete or reinforced-concrete arches is that centering for the latter types of arches serves as a mold for shaping the soffit of the arch ring, the face of the arch ring, and the spandrel walls.

The successful construction of arches depends nearly as much on the centers and their supports as it does on the design of the arch. The centers should be as well constructed and the supports as unyielding as it is possible to make them. When it is necessary to use piles,

when it is necessary to span a stream or roadway. Sometimes the length of the span for the centering is very short, or there are a series of short spans, or the span may be equal to that of the arch. The trusses must be carefully designed, in order that the deflection and deformation due to the changes in the loading will be reduced to a minimum. By placing a temporary load on the centers at the crown, the deformation during construction may be very greatly reduced. This load is removed as the weight of the arches comes on the centers. For the design of trusses, the reader is referred to the Instruction Papers, or other treatises, on Bridge Engineering and Roof Trusses.

The lagging for concrete arches usually consists of 2- by 3-inch or 2- by 4-inch plank, either set on edge or laid flat, depending on the thickness of the arch and the spacing of the supports. The surface on which the concrete is laid is usually surfaced on the side on which the concrete is to be placed. The lagging is very often supported on ribs constructed of 2- by 12-inch plank, on the back of which is placed a 2-inch plank cut to a curve parallel with the intrados. These 2- by 12-inch planks are set on the timber used to cap the piles, and are usually spaced about 2 feet apart. All the supports should be well braced. The centers should be constructed to give a camber to the arch about equal to the deflection of the arch when under full load. It is, therefore, necessary to make an allowance for the settlement of centering, for the deflection of the arch after the removal of the centering, and for permanent camber.

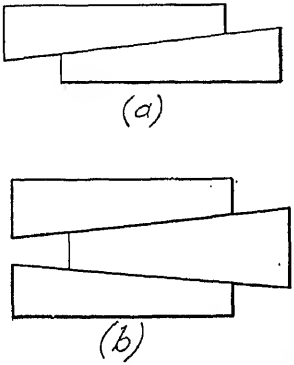


Fig. 172. Wedges Used in Placing and Removing Forms

The centers should be constructed so that they can easily be taken down. To facilitate the striking of centers, the practice is to support them on folding wedges or sand boxes. When the latter method is used, the sand should be fine, clean, and perfectly dry, and the boxes should be sealed around the plunger with cement mortar. Striking forms by means of wedges is the commoner method. The type of wedges generally used is shown in Fig. 172-*a*, although sometimes three wedges are used, as shown by Fig. 172-*b*. They are from 1 to 2 feet long, 6 to 8 inches wide, and have a slope of from 1:6

TABLE XXVII*

Safe Load in Pounds Uniformly Distributed for Rectangular Beams, One Inch Thick, Long-Leaf Yellow Pine

Allowable fiber stress, 1,200 pounds per square inch; factor of safety, 6; modulus of rupture, 7,200 pounds per square inch

Safe loads for other factors of safety may be obtained as follows:

$$\text{New safe load} = \text{Safe load from table} \times \frac{6}{\text{New factor}}$$

Span in Feet	Depth of Beam in Inches									Deflection Coefficient
	4	5	6	7	8	10	12	14	16	
4	533	833	1,200	1,633	2,133	3,333	4,800	6,533		.38
5	427	667	960	1,307	1,707	2,667	3,840	5,227		.60
6	356	556	800	1,089	1,422	2,222	3,200	4,356		.86
7	305	476	686	933	1,219	1,905	2,743	3,733		1.18
8	267	417	600	817	1,067	1,667	2,400	3,267		1.54
9	237	370	533	726	948	1,481	2,133	2,904	3,793	1.94
10	213	333	480	653	853	1,333	1,920	2,613	3,413	2.40
12	178	278	400	544	711	1,111	1,600	2,178	2,844	3.46
14	152	238	343	467	610	952	1,371	1,867	2,438	4.70
16	133	208	300	408	533	833	1,200	1,633	2,133	6.14
18	119	185	267	363	474	741	1,067	1,452	1,896	7.78
20	107	167	240	327	427	667	960	1,307	1,707	9.60
22	97	152	218	297	388	606	873	1,188	1,552	11.62
24	89	139	200	272	356	556	800	1,089	1,422	13.82
26		128	185	251	328	513	738	1,005	1,313	16.22
28		119	171	233	305	476	686	933	1,219	18.82
30		111	160	218	284	444	640	871	1,138	21.60

To find the safe load for beams of hemlock from Table XXVII, the above values must be divided by 2; for beams of short-leaf yellow pine and white oak, the values must be divided by 1.2; for white pine, spruce, eastern fir, and chestnut, the values must be divided by 1.71.

to 1:10. The centering is lowered by driving back the wedges; and to do this slowly, it is necessary that the wedges have a very slight taper. All wedges should be driven equally when the centering is being lowered. The wedges should be made of hardwood, and are placed on top of the vertical supports or on timbers which rest on the supports. The wedges are placed at about the same elevation as the springing line of the arch.

Tables XXVII and XXVIII can be used to assist in the design of the different members of the centers for arches.

Safe Stresses in Lumber for Wood Forms. In Table XXVII are given the safe loads which may be placed on beams of long-leaf yellow pine, of various depths, on various spans.

TABLE XXVIII*

Strength of Solid Wood Columns of Different Kinds of Timber

	White Oak, Southern Long-Leaf Pine	Douglas Fir Short-Leaf Pine	Red Pine (Norway Pine), Spruce (Eastern Fir), Hemlock Cypress, Chestnut California Redwood, California Spruce	White Pine Cedar
F	5,000	4,500	4,000	3,500
$\frac{l}{d}$				
4	4,897	4,407	3,918	3,428
6	4,782	4,304	3,826	3,347
8	4,638	4,174	3,710	3,247
10	4,474	4,026	3,579	3,132
12	4,297	3,867	3,438	3,008
14	4,114	3,703	3,291	2,880
16	3,930	3,537	3,144	2,751
18	3,748	3,373	2,998	2,624
20	3,571	3,214	2,857	2,500
22	3,402	3,061	2,721	2,381
26	3,086	2,777	2,469	2,160
30	2,805	2,524	2,244	1,963
36	2,445	2,200	1,956	1,711
40	2,241	2,017	1,793	1,569
50	1,835	1,652	1,468	1,285

To find the load that a wood column will support per square inch of sectional area, from Table XXVIII, the length of the column in inches is divided by the least diameter of the column, and the result is the ratio of length to diameter of the column. From this ratio is found the ultimate strength per square inch of section of a column of any kind of wood given in Table XXVIII. A factor of safety of 5 should be used in finding the size of column required; that is, the working load should not be greater than one-fifth of the values given.

uniformly distributed, exclusive of the weight of the beam itself, for rectangular beams one inch thick. The safe load for a beam of any thickness may be found by multiplying the values given in the tables by the thickness of the beam in inches. From the last column, the deflection may be obtained, corresponding to the given span and safe load, by dividing the coefficient by the depth of the beam in inches, which will give approximately the deflection in inches.

Example. If a beam is required to support a uniformly distributed load of 4,000 pounds on a span of 10 feet, what would be the dimensions of the beam of long-leaf yellow pine, and what would be the deflection?

Solution. Following the line for beams of 10-foot span, it is found that a beam 8 inches deep and 5 inches wide ($853 \times 5 = 4{,}265$) would support the load of 4,000 pounds, and the deflection would be $2.40 \div 8 = .30$ inch. A second

solution would be to use a beam 12 inches deep and 2 inches wide ($1,920 \times 2 = 3,840$); but according to Table XXVII this beam would not be quite strong enough, as it would only support a load of 3,840 pounds.

Safe Loads on Wood Columns. The values given in Table XXIX are based on the formula:

$$P = F \times \frac{700 + 15c}{700 + 15c + c^2}$$

where P is the ultimate strength of timber in pounds per square inch; F is the ultimate crushing strength of timber; l is the length of column, in inches; d is the least diameter in inches; and c equals $\frac{l}{d}$.

Example. If a column 10 feet long is required to support a load of 20,000 pounds, what would be the size of the column required if California redwood were used?

Solution. Dividing the length of the beam in inches by the assumed least diameter, 6 inches, we have $120 \div 6 = 20$, which gives the ratio of the length to the diameter. By Table XXVIII it is shown that 2,857 pounds is the ultimate strength for a column of California redwood, when $l \div d = 20$. Assuming a factor of safety of 5, and dividing 2,857 by 5, the working load is found to be 571 pounds per square inch. Dividing 20,000 by 571, it is found that a column whose area is 35 square inches is required to support the load. The square root of 35 is 5.9. Therefore a column of California redwood 6 inches square will support the load.

Form for Arch at 175th Street, New York City. In constructing the 175th Street Arch in New York City, the forms were so built that they could be easily moved. The arch is elliptical and is built of hard-burned brick and faced with granite. The span of the arch is 66 feet; the rise is 20 feet; the thickness of the arch ring is 40 inches and 48 inches, at the crown and the springing line, respectively; and the arch is built on a 9-degree skew. The total length of this arch is 800 feet.

The arch is constructed in sections, the centering being supported on 11 trusses placed perpendicular to the axis of the arch and having the form and dimensions shown in Fig. 173. The trusses are placed 5 feet on centers, and are supported at the ends and middle by three lines of 12- by 12-inch yellow pine caps. The caps are supported by 12- by 12-inch posts, spaced five feet center to center, and rest on timber sills on concrete foundations. The upper and

is $2\frac{3}{4}$- by 6-inch, long-leaf yellow pine plank. The connections of the timbers are made by means of $\frac{3}{8}$-inch steel plates and $\frac{7}{8}$-inch bolts, arranged as shown in the illustration. As it was absolutely necessary

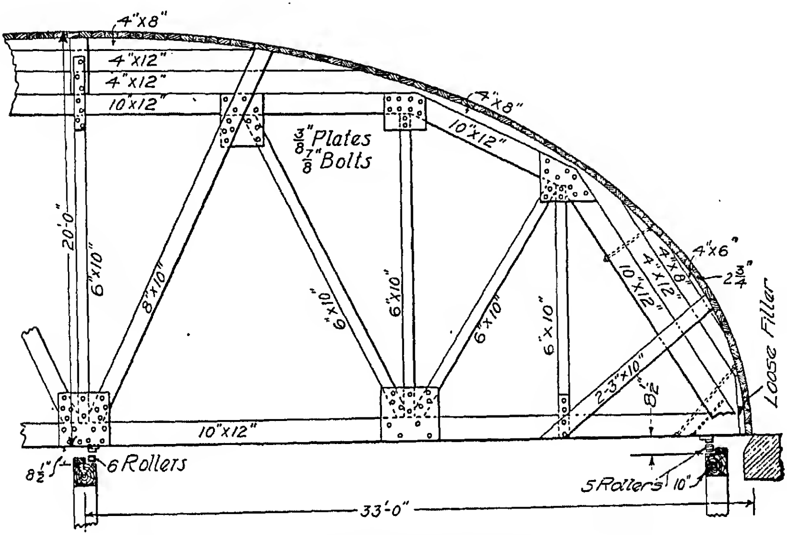


Fig. 173. Typical Arch Form Used at 175th Street, New York City

to have the forms alike, to enable them to be moved along the arch and at all times fit the brickwork, they were built on the ground from the same pattern, and hoisted to their places by two guyed derricks with 70-foot booms.

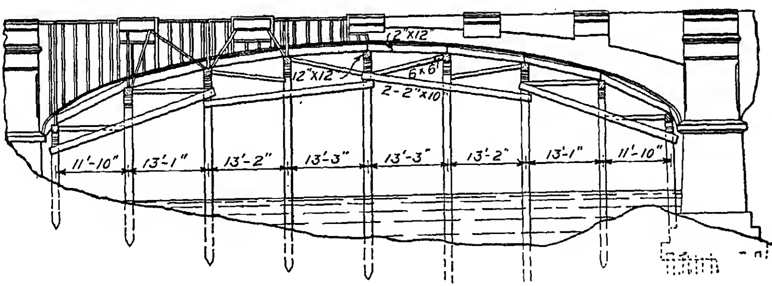


Fig. 174. Centers for Bridge at Canal Dover, Ohio

On the 12- by 12-inch cap was a 3- by 8-inch timber, on which the double wedges were placed. When it was necessary to move the forms, the wedges were removed, the forms rested on the rollers, and

there was then a clearance of about $2\frac{1}{4}$ inches between the brick-work and the lagging. The timber on which the rollers ran was faced with a steel plate $\frac{1}{4}$ inch by 4 inches in dimensions. The forms were moved forward by means of the derricks. The settlement of the forms under the first section constructed was $\frac{1}{4}$ inch; and the settlement of the arch ring of that section, after the removal of forms, was $\frac{1}{4}$ inch.*

Forms for Bridge at Canal Dover, Ohio.* The details of the centering used in erecting one of the 106-foot 8-inch spans of a reinforced-concrete bridge over the Tuscarawas River at Canal Dover, Ohio, are shown in Figs. 174 and 175. Besides this span, the bridge consisted of two other spans of 106 feet 8 inches each, and a

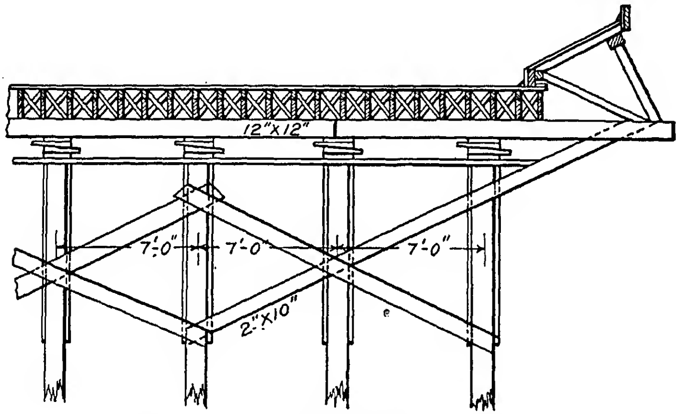


Fig. 175. Centers for Bridge at Canal Dover, Ohio

canal span of 70 feet. The centering for the canal span was built in 6 bents, each bent having 7 piles. A clear waterway of 18 feet was required in the canal span by the State Canal Commissioner, and this passage was arranged under the center of the arch. The piles were driven by means of a scow. The cap for the piles was a 3- by 12-inch timber. Planks 2 inches thick were sawed to the correct curvature, and nailed to the 2- by 12-inch joists, which were spaced about 12 inches apart. The lagging was one inch thick, and was nailed to the curved plank. The wedges were made and used as shown. The centering was constantly checked; this was found important after a strong wind. The centering for the other two of the main arches was constructed similarly to that of the arch shown.

After some difficulty had been experienced in keeping the forms in place during the concreting of the first arch, the concrete for the other arches was placed in the order shown in Fig. 176, and no difficulty was encountered. Sections 1 and 1 were first placed, then 2 and 2, etc., finishing with section 6.

The concreting on the canal span was begun in the late fall, and

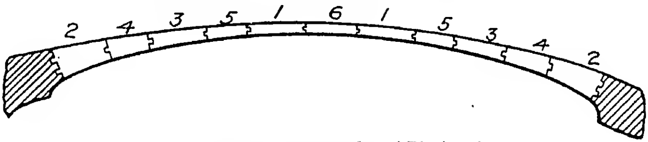


Fig. 176. Diagram Showing Order of Placing Concrete in Bridge at Canal Dover, Ohio

finished in 12 days; the forms were lowered by means of the wedges five weeks later. The deflection at the crown was 0.5 inch, and after the spandrel walls were built and the fill made, there was an additional deflection of 0.4 inch. In building the forms, an allowance of $\frac{1}{800}$ part of the span was made, to allow for this deflection. The deflections at the crown of the other three arches were 0.6 inch, 1.45 inches, and 1.34 inches, respectively.

BENDING OR TRUSSING BARS

Bending Details. Drawings showing all the bending details of the bars, for all reinforced-concrete work, should be made before the steel is ordered. The designing engineer should detail a few of the

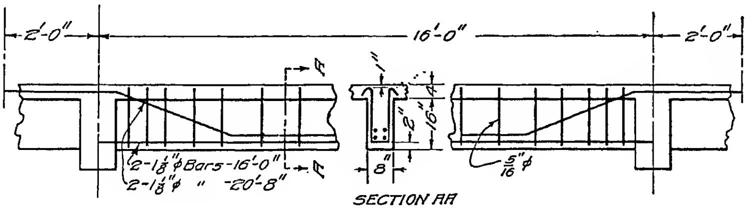


Fig. 177. Details of Beam Construction

typical beams and girders to show, in a general way, what length of bars will be required, the number of turned-up bars, the number, size, and spacing of stirrups required, and the dimensions of the concrete. These details will then be a guide for the construction engineer to make up the details required to properly construct the

should detail a typical beam so that the constructing engineer can develop these details as shown in Fig. 178.

Tables for Bending Bars. A simple outfit for bending the bars cold consists of a strong table, the top of which is constructed as shown in Fig. 179. The outline to which the bar is to be bent is laid out on

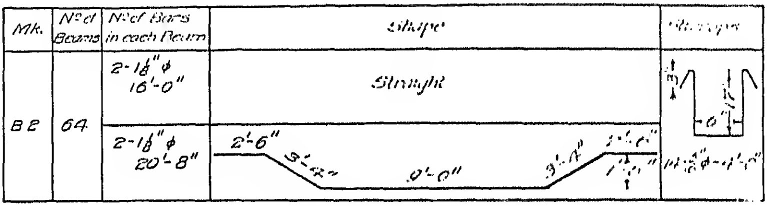


Fig. 178. Bending Details for Beams

the table, and holes are bored at the point where the bends are to be made. Steel plugs 5 inches to 6 inches long are then placed in these holes. Short pieces of boards are nailed to the table where necessary, to hold the bar in place while being bent. The bar is then placed in

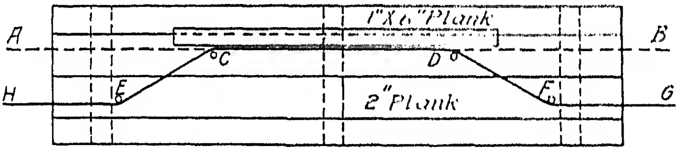


Fig. 179. Plan of Bending Table

the position *A-B*, Fig. 176, and bent around the plugs *C* and *D*, and then around the plugs *E* and *F*, until the ends *EH* and *FG* are parallel to *AB*. When bends with a short radius are required, the bars are placed in the vise, near the point where the bend is wanted,

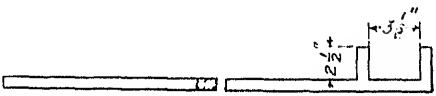


Fig. 180. Type of Lever Bender

and the end of the bar is pulled around until the required angle is secured. The vise is usually fastened to the table. The lever shown in Fig. 180 is also used in making bends of short radii. This is done

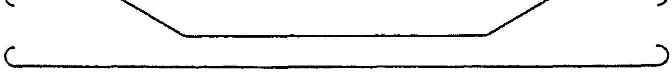


Fig. 181. Bars with Hooked Ends

ends of all the bars in the beams and girders shall be hooked as shown in Fig. 181. This is done to prevent the bars from slipping before their tensile strength is fully developed.

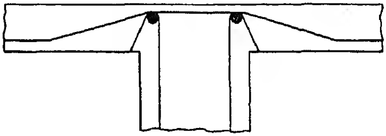


Fig. 182. Slab Bars

Slab Bars. To secure the advantage of a continuous slab, it is very often required that a percentage of the slab bars, usually one-half, shall be turned up over each beam. Construction companies have different methods of bending and holding these bars in place; but the method shown in Fig. 182 will insure good results, as

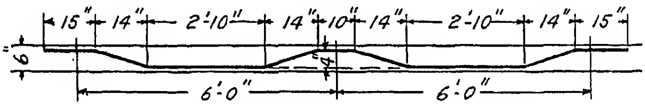


Fig. 183. Diagram Showing Bent Bars for Slabs

the slab bars are well supported by the two longitudinal bars which are wired to the tops of the stirrups. Fig. 183 shows the bending details of slab bars, the beams being spaced six feet, center to center. When slabs are designed as simple beams $\left(\frac{Wl}{8}\right)$ none of the slab bars are bent.

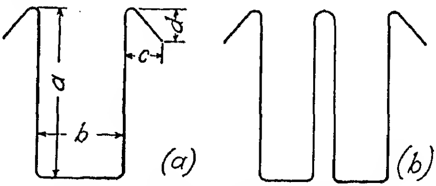


Fig. 184. Diagram Showing Bending Bars for Stirrup

Stirrups. Fig. 184 shows the bending of the bars for stirrups. The ends of the stirrups rest on the forms and support the beam bars, which assist in keeping these bars in place. The ends of the stirrups never show on the bottom of the slab of the finished floor, although the cut ends of the stirrups rest directly on the slab forms. Sufficient mortar seems to get under the ends of the

stirrups to cover them. The type of stirrup shown in Fig. 184-*a* is much more extensively used than that in Fig. 184-*b*. The latter type is most frequently used when a large amount of steel is required for stirrups, or if the stirrups are made of very small bars.

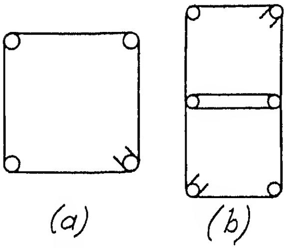


Fig. 185. Column Bands

Column Bands. In Fig. 185 two types of column bands are shown. Fig. 185-*a* shows bands for a square or a round column; and Fig. 185-*b*, bands for a rectangular column. The bar which forms the band is bent close around each vertical bar in the columns, and therefore assists in holding these bars in place. The bands for the rectangular column *b* are made up of two separate bands of the same size and shape.

Spacers. Spacers for holding the bars in place in beams and girders have been successfully used. These spacers, Fig. 186, are made of heavy sheet iron. They are fastened to the stirrups by means of the loops in the spacers. The ends of the spacers which project out to the forms of the sides of the beams should be made blunt or rounded. This will prevent the ends of the spacers being driven into the forms when the concrete is being tamped. The number of these spacers required will depend on the lengths of the beams; usually 2 to 4 spacers are used in each beam.

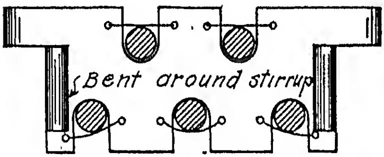


Fig. 186. Typical Spacer for Reinforcing Bars

Several devices have been manufactured for holding slab bars in place while the concrete is being poured. Fig. 187 shows a spacer, patented and manufactured by the Concrete Steel Company, that has been in use for several years and has been found satisfactory.

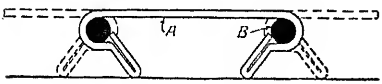


Fig. 187. Spacer for Slab Bars

Unit Frames. Companies making a specialty of supplying

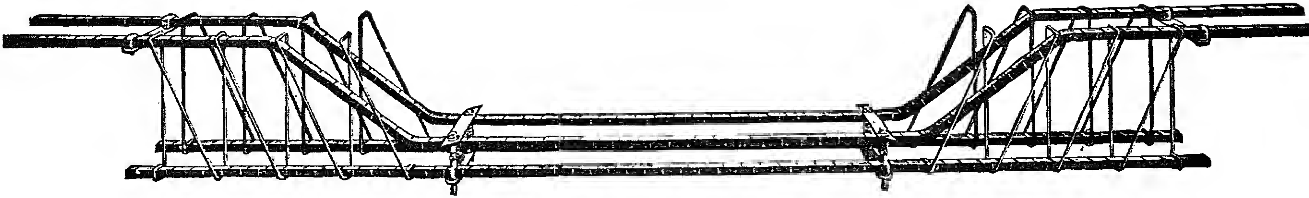


Fig. 188. Reinforcing Steel Bars Made into a Unit
Courtesy of Corrugated Bar Company, Buffalo, New York

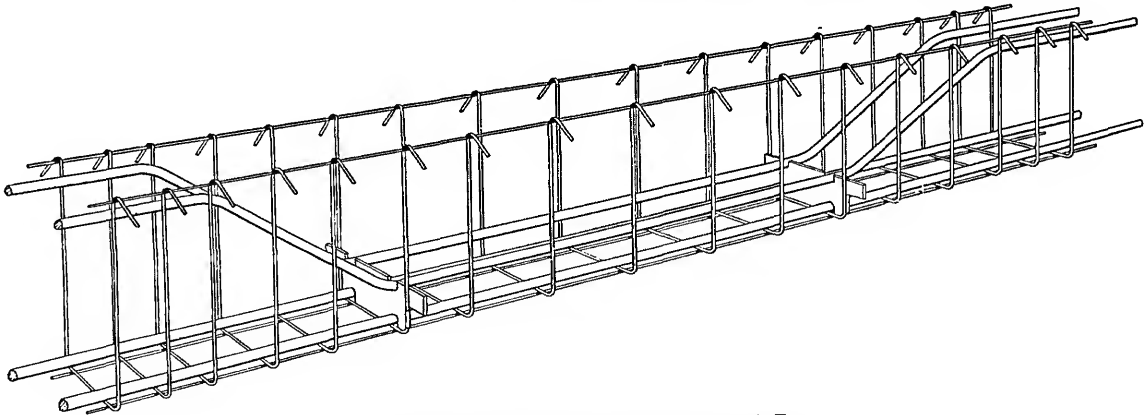


Fig. 189. Design of Collapsible Reinforcing Bar Unit Frame
Courtesy of Concrete Steel Company, Philadelphia, Pennsylvania

work, and shipped to the job as a unit. Fig. 188 shows a unit made by the Corrugated Bar Company, in which the shear bars are laced around the tension bars. These units can be closed up for shipment.

Fig. 189 shows a collapsible frame made by the Concrete Steel Company. The frame is made up of four small bars, usually $\frac{1}{4}$ inch around, and the stirrups that are required for the beam are fastened to these bars by clips that will permit the frame to be folded up for shipment. When the frame is received on the job it is unfolded, placed in the beam for which it is designed, and then the tension bars are put in the frame and held in place by two or more spacers.

BONDING OLD AND NEW CONCRETE

The place and manner of making breaks or joints in floor construction at the end of a day's work is a subject that has been much discussed by engineers and construction companies. But there has not yet been any general agreement as to the best method and place of constructing these joints. Wherever joints are made, great care should be exercised to secure a bond between the new and the old concrete.

Methods of Making Bonds. *First Method.* Fig. 190 shows a sectional view of one method of making a break at the end of the day's work; this method has been used very extensively and successfully. The stirrups and slab bars form the main bond between the old and the new work, if the break is left more than a few hours. Short bars in the top of the slab will also assist in making a good bond; an additional number of stirrups should also be used where the break is to be made in the beam. Before the new concrete is placed, the old concrete should be well scraped, thoroughly soaked with clean water,

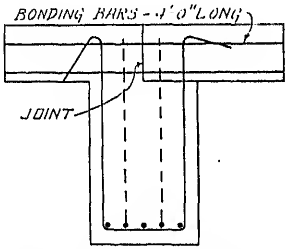


Fig. 190. Method of Bonding Old and New Concrete in Slab

Second Method. Another method of forming stopping places is by dividing the beam vertically—that is, making two L-beams instead of one T-beam, Fig. 191. Theoretically, this is a very good method, but practically, it is found difficult to construct the forms dividing the beam, as the steel is greatly in the way.

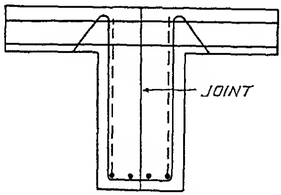


Fig. 191. Method of Bonding Old and New Concrete in Beam

Third Method. The method of stopping the work at the center of the span of the beams and parallel to the girders is the method in general use. Fig. 192 illustrates this method. Theoretically, the slab is not weakened; and as the maximum bending moment occurs at this point, the shear is zero, and, therefore, the beams are not supposed to be weakened, except for the loss of concrete in tension, and this is not considered in the calculation. The bottoms of the beams are tied together by the steel that is placed in the beams to take the tensile stresses; and there should be some short bars placed in the top of these beams, as well as in the top of the slab, to tie them together. The objection made in the description of the first method—in that any shrinkage in the concrete at the joint will permit water to pass through—is greater in the second and third methods than in the first.

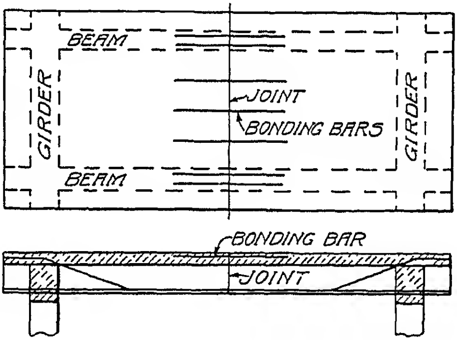


Fig. 192. Method of Bonding Break in Center of Span

FINISHING SURFACES OF CONCRETE

Imperfections. To give a satisfactory finish to exposed surfaces of concrete is a rather difficult problem. In many instances, when the forms are taken down, the surface of the concrete shows the joints, knots, and grain of the wood; it has more the appearance of a piece of rough carpentry work than that of finished masonry. Also, failure to tamp or flat-spade the surfaces next to the forms will result in

will cause a variation in the surface texture of the concrete. Variation of color, or discoloration, is one of the most common imperfections. Old concrete adhering to the forms will leave pits in the surface; or the pulling-off of the concrete in spots, as a result of its adhering to the forms when they are removed, will cause a roughness.

To guard against these imperfections, the forms must be well constructed of dressed lumber, and the pores should be well filled with soap or paraffine. The concrete should be thoroughly mixed, and, when placed, care should be taken to compact the concrete thoroughly, next to the forms. The variation in color is usually due to the leaching-out of lime, which is deposited in the form of an efflorescence on the surface; or to the use of different cements in adjacent parts of the same work. The latter cause can almost always be avoided by using the same brand of cement on the entire work, and the former will be treated under the heading of "Efflorescence".

Plastering. Plastering is not usually satisfactory, although there are cases where a mixture of equal parts of cement and sand has, apparently, been successful; and, when finished rough, it did not show any cracks. It is generally considered impossible to apply mortar in thin layers to a concrete surface, and make it adhere for any length of time. When the plastering begins to scale off, it looks worse than the unfinished surface. This paragraph is intended more as a warning against this manner of finishing concrete surfaces than as a description of it as an approved method of finish.

Mortar Facing. The following method has been adopted by the New York Central Railroad for giving a good finish to exposed concrete surfaces:

The forms of 2-inch tongued-and-grooved pine were coated with soft soap, all openings in the joints of the forms being filled with hard soap. The concrete was then deposited, and, as it progressed, was drawn back from the face with a square-pointed shovel, and 1:2 mortar poured in along the forms. When the forms were removed, and while the concrete was green, the surface was rubbed, with a circular motion, with pieces of white fire brick, or brick composed of one part cement and one part sand. The surface was then dampened

A method of placing mortar facing that has been found very satisfactory, and has been adopted very extensively in the last few years, is as follows: A sheet-iron plate, 6 or 8 inches wide and about 5 or 6 feet long, has riveted across it on one side, angles of $\frac{3}{4}$-inch size, or such other size as may be necessary to give the desired

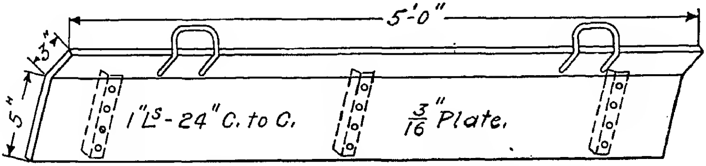


Fig. 193. Sheet-Iron Plate for Giving Finish Surface to Concrete

thickness of mortar facing, these angles being spaced about two feet apart, Fig. 193. In operation, the ribs of the angles are placed against the forms; and the space between the plate and forms is filled with mortar, which is mixed in small batches, and thoroughly tamped. The concrete back filling is then placed; the mold is withdrawn; and the facing and back filling are rammed together. The mortar facing is mixed in the proportion of one part cement, to 1, 2, or 3 parts sand; usually a 1:2 mixture is employed, mixed wet and in small batches as it is needed for use. As mortar facing shows the roughness of the forms more readily than concrete does, care is required, in constructing, to secure a smooth finish. When the forms are removed, the face may be treated either in the manner already described, or according to the following method taken from the Proceedings of the American Railway Engineering Association:

After the forms are removed, any small cavities or openings in the concrete shall be filled with mortar, if necessary. Any ridges due to cracks or joints in the lumber shall be rubbed down; the entire face shall be washed with a thin grout of the consistency of whitewash, mixed in the proportion of 1 part cement to 2 parts of sand. The wash shall be applied with a brush.

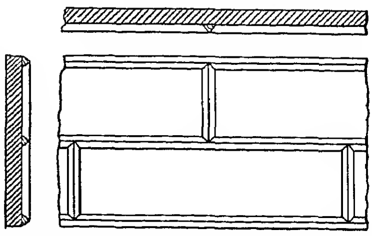


Fig. 194. Diagram Showing Method of giving Masonry Facing to Concrete

fastened to the forms to make V-shaped depressions in the concrete, as shown in Fig. 194.

Stone or Brick Facing. A facing of stone or brick is frequently used for reinforced concrete, and is a very satisfactory solution of the problem of finish. The same care is required with a stone or brick facing as if the entire structure were stone or brick. The Ingalls Building at Cincinnati, Ohio, 16 stories, is veneered on the outside with marble to a height of three stories, and with brick and terra cotta above the third story. Exclusive of the facing, the wall is 8 inches thick.

In constructing the Harvard University Stadium, care was taken, after the concrete was placed in the forms, to force the stones back from the face and permit the mortar to cover every stone. When the forms were removed, the surface was picked with the tool shown in Fig. 195. A pneumatic tool has also been adopted for this purpose.

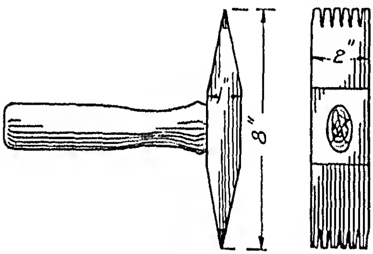
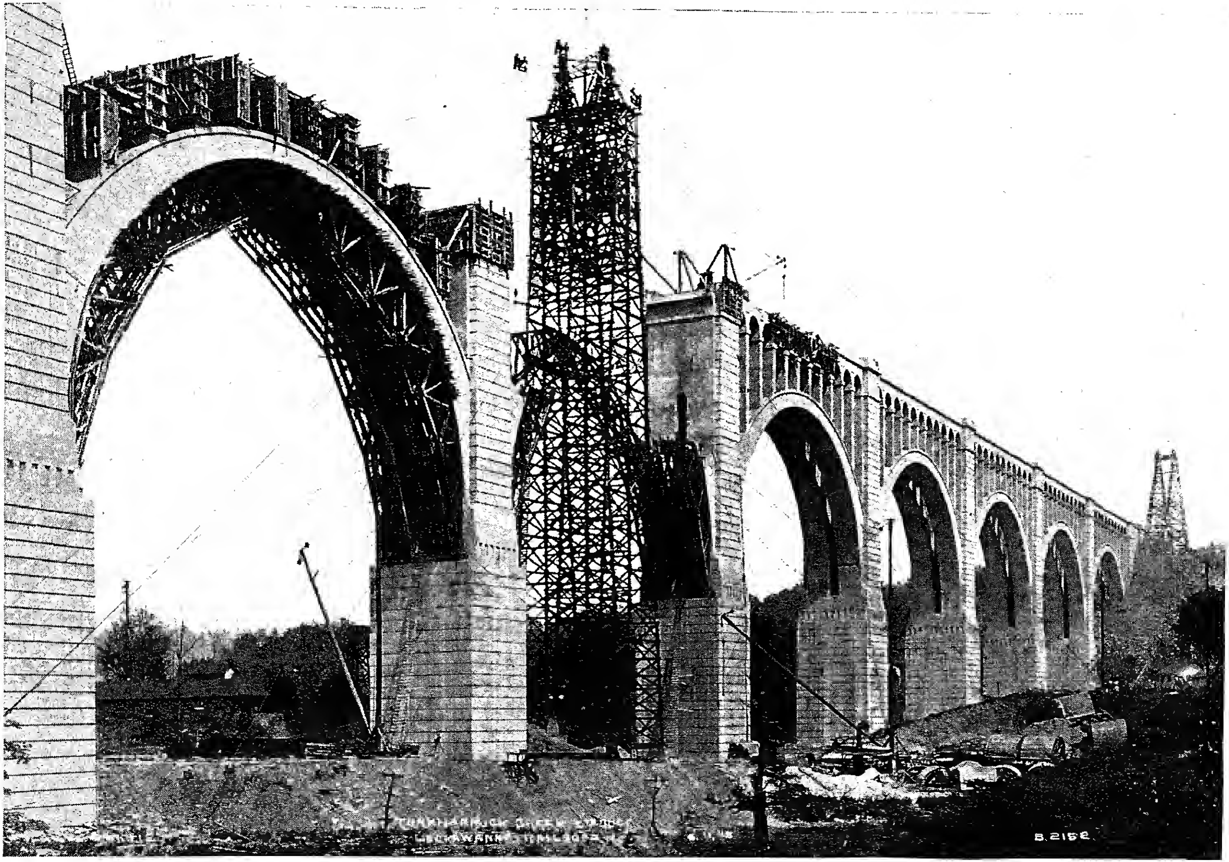


Fig. 195. Typical Facing Hammer

The number of square feet to be picked per day depends on the hardness of the concrete. If the picking is performed by hand, it is done by a common laborer; and he is expected to cover, on an average, about 50 square feet per day of 10 hours. With a pneumatic tool, a man would cover from 400 to 500 square feet per day.

Recently a motor-driven hand tool, Fig. 196, has been invented. This works dry, and it leaves the surface slightly porous, so that it provides an excellent base for the application of a float or a coat of paint. The machine is driven through a flexible shaft by a motor carried by the operator. The whole apparatus, motor included, weighs only 20 pounds. The motor may take its actuating current from an ordinary electric light socket.

The method of chipping the concrete surface is very ingenious. Mounted in a disk are twenty-four cutter wheels arranged in pairs, each wheel having from twenty-four to twenty-eight cutting teeth. As the disk revolves at high speed, the cutter wheels are made to roll over the concrete surface, each tooth acting as a tiny hammer



REINFORCED CONCRETE ARCH CONSTRUCTION USED ON TUNKHANNOCK CREEK VIADUCT

Courtesy of Engineering Department, Delaware, Lackawanna and Western Railroad

but instead of being radial, the teeth are eccentrically directed so that their edges are brought into contact absolutely square with the surface, and deal a direct blow to the material that is to be cut. The disk revolves at the rate of about 2,000 revolutions a minute, so that the number of blows per minute delivered by the cutter wheels runs up into the millions. The cutting tool is of such a form that it may be conveniently grasped and guided by the operator, and on it is a small switch by means of which the power may be readily turned on and off. The average work per day of this tool is from 700 to 900 square feet. It may be used, as well, for surfacing stone and imitation stone, and for bringing out the aggregate in concrete, when that is desired.

Fig. 196. Power-Driven Hand Tool for Surfacing Concrete
Courtesy of "Scientific American"

Granolithic Finish. Several concrete bridges in Philadelphia have been finished according to the following specifications and their appearance is very satisfactory:

Granolithic surfacing, where required, shall be composed of 1 part cement, 2 parts coarse sand or gravel, and 2 parts granolithic grit, made into a stiff mortar. Granolithic grit shall be granite or trap rock, crushed to pass a $\frac{1}{4}$-inch sieve, and screened of dust. For vertical surfaces, the mixture shall be deposited against the face forms to a minimum thickness of 1 inch, by skilled workmen, as the placing of the concrete proceeds; and it thus forms a part of the body of the work. Care must be taken to prevent the occurrence of air space or voids in the surface. The face shall be removed as soon as the concrete has sufficiently

hardened; and any voids that may appear shall be filled with the mixture. The surface shall then be immediately washed with water until the grit is exposed and rinsed clean, and shall be protected from the sun and kept moist for three days. For bridge-seat courses and other horizontal surfaces, the granolithic mixture shall be deposited on the concrete to a thickness of at least $1\frac{1}{2}$ inches, immediately after the concrete has been tamped and before it has set, and shall be troweled to an even surface, and, after it has set sufficiently hard, shall be washed until the grit is exposed.

The success of this method depends greatly on the removal of the forms at the proper time. In general, the washing is done the day following that on which the concrete is deposited. The fresh concrete is scrubbed with an ordinary scrubbing brush, removing the film, and the impressions of the forms, and exposing the sand and stone of the concrete. If this is done when the material is at the

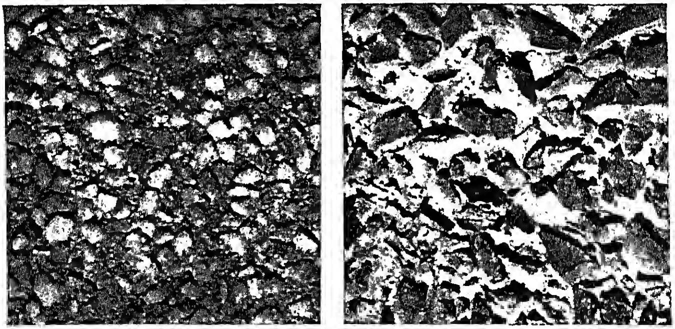


Fig. 197. Quimby's Finish on Concrete Surfaces. Left—Aggregate $\frac{3}{16}$ Inch White Pebbles; Right—Aggregate $\frac{3}{8}$ Inch Screened Stone

proper degree of hardness, merely a few rubs of an ordinary house scrubbing brush, with a free flow of water to cut and to rinse clean, constitutes all the work and apparatus required. The cost of scrubbing is small if done at the right time. A laborer will wash 100 square feet in an hour; but if that same area is permitted to get hard, it may require two men a day, with wire brushes, to secure the desired results. The practicability of removing the forms at the proper time for such treatment depends upon the character of the structure and the conditions under which the work must be done. This method is applicable to vertical walls, but it would not be applicable to the soffit of an arch, Fig. 197.

The Acid Treatment. This treatment consists in washing the surface of the concrete with diluted acid, then with an alkaline

expose the sand and stone; the alkaline solution is then applied to remove all of the free acid; and, finally, the surface is washed with clear water. The treatment may be applied at any time after the forms are removed; it is simple and effective. Limestone cannot be used in the concrete for any surfaces that are to have this treatment, as the limestone would be affected by the acid. This process has been used very successfully.

Dry Mortar Finish. The dry mortar method consists of a dry, rich mixture, with finely crushed stone. The concrete is usually composed of 1 part cement, 3 parts sand, and 3 parts crushed stone, known as the $\frac{1}{4}$-inch size, and mixed dry so that no mortar will flush to the surface, when well rammed in the forms. When placed, the concrete is not spaded next to the forms and, being dry, there is no smooth mortar surface, but there should be an even-grained, rough surface. With the dry mixture, the imprint of joints of the forms is hardly noticed, and the grain of the wood is not seen at all. This style of finish has been extensively used in the South Park system of Chicago, and there has been no efflorescence apparent on the surface, which is explained by "the dryness of the mix and the porosity of the surface".

Cast Slab Veneer. Cast-concrete-slab veneer can be made of any desired thickness or size. It is set in place like stone veneer, with the remainder of the concrete forming the backing. It is usually cast in wood molds, face down. A layer of mortar, 1 part cement, 1 part sand, and 2 or 3 parts fine stone or coarse sand is placed in the mold to a depth of about 1 inch, and then the mold is filled up with a 1:2:4 concrete. Any steel reinforcement that is desired may be placed in the concrete. Usually, cast-concrete-slab veneer is cheaper than concrete facing cast in place, and a better surface finish is secured by its use.

Moldings and Ornamental Shapes. Concrete is now in demand in ornamental shapes for buildings and bridges. The shapes may be either constructed in place, or molded in sections and placed the same as cut stone. Plain cornices or panels are usually constructed in place, but complicated molding or balusters, Fig. 198, are frequently made in sections and erected in separate pieces.

The molds may be constructed of wood, metal, or plaster of Paris, or molded in sand. The operation of casting concrete in sand

TABLE XXIX*

Colors Given to Portland Cement Mortars Containing Two Parts River Sand to One Part Cement

Dry Material Used	Weight of Dry Coloring Matter to 100 Lb. of Cement				Cost of Coloring Matter per Pound
	½ Pound	1 Pound	2 Pounds	4 Pounds	
Lampblack	Light Slate	Light Gray	Blue-Gray	Dark Blue Slate	15 cents
Prussian Blue	Light Green Slate	Light Blue Slate	Blue Slate	Bright Blue Slate	50 cents
Ultramarine Blue	....	Light Blue Slate	Blue Slate	Bright Blue Slate	20 cents
Yellow Ocher	Light Green	....	....	Light Buff	3 cents
Burnt Umber	Light Pinkish Slate	Pinkish Slate	Dull Lavender-Pink	Chocolate	10 cents
Venetian Red	Slate, Pink Tinge	Bright Pinkish Slate	Light Dull Pink	Dull Pink	2½ cents
Red Iron Ore	Pinkish Slate	Dull Pink	Terra Cotta	Light Brick Red	2½ cents

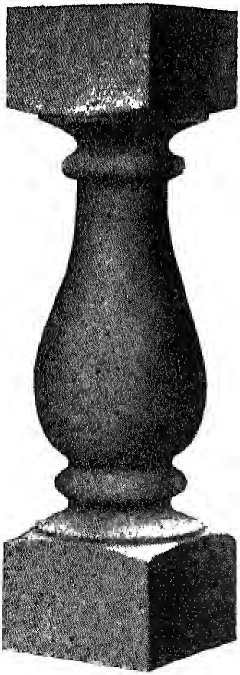


Fig. 198. Typical Molded

is similar to that of casting iron. The pattern is made of wood the exact size required. It is then molded in flasks exactly as is done in casting iron. The ingredients for concrete consist of cement and sand or fine crushed stone; the mixture, with a consistency about that of cream, is poured into the mold with the aid of a funnel and a T-pipe. Generally, the casting is left in the sand for three or four days, and, after being taken out of the sand, should harden in the air a week or ten days before being placed. Balusters are very often made in this manner.

Colors for Concrete Finish. Coloring matter has not been used very extensively in concrete work, except in ornamental work. It has not been very definitely determined what coloring matters are detrimental to concrete. Lampblack (boneblack) has been used more extensively than any other coloring mat-

ing on the amount used. Common lampblack and Venetian red should not be used, as they are apt to run or fade. Dry mineral colors, mixed in proportions of 2 to 10 per cent of the cement, give shades approaching the color used. Red lead should never be used; even one per cent is injurious to the concrete. Variations in the color of cement and in the character of the sand used will affect the results obtained in using coloring matter as shown in Table XXIX.

Painting Concrete Surfaces. Special paints are made for painting concrete surfaces. Ordinary paints, as a rule, are not satisfactory. Before the paint is applied, the surface of the wall should be washed with dilute sulphuric acid, 1 part acid to 100 parts water.

Finish for Floors. Floors in manufacturing buildings are often finished with a 1-inch coat of cement and sand, mixed in the proportions of 1 part cement to 1 part sand; or 1 part cement to 2 parts

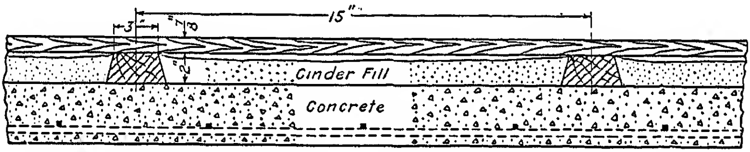


Fig. 199. Diagram Showing Typical Cinder Fill between Stringers

sand. This finishing coat must be put on before the concrete base sets, or it will break up and shell off, unless it is made very thick, $1\frac{1}{2}$ to 2 inches. A more satisfactory method of finishing such floors is to put 2 inches of cinder concrete on the concrete base, and then put the finishing coat on the cinder concrete. The finish coat and cinder concrete bond together, making a thickness of 3 inches. The cinder concrete may consist of a mixture of 1 part cement, 2 parts sand, and 6 parts cinders, and may be put down at any time; that is, this method of finishing a floor can be used as satisfactorily on an old concrete floor as on one just constructed.

In office buildings, and generally in factory buildings, a wood floor is laid over the concrete. Wood stringers are first laid on the concrete, about 1 to $1\frac{1}{2}$ feet apart. The stringers are 2 inches thick and 3 inches wide on top, with sloping edges. The space between the stringers is filled with cinder concrete, as shown in

Fig. 199; as a rule this is mixed 1:4:8. When the concrete has set, the flooring is nailed to the stringers. Usually a layer of waterproof paper or saturated felt is spread between the concrete and the flooring to prevent the floor from warping.

Efflorescence. The white deposit found on the surface of concrete, brick, and stone masonry is called efflorescence. It is caused by the leaching of certain lime compounds, which are deposited on the surface by the evaporation of the water. This is believed to be due, primarily, to the variation in the amount of water used in mixing the mortar. An excess of water will cause a segregation of the coarse and fine materials, resulting in a difference of color. In a very wet mixture, more lime will be set free from the cement and brought to the surface. When great care is used as to the amount of water, and care is taken to prevent the separation of the stone from the mortar when deposited, the concrete will present a fairly uniform color when the forms are removed. There is greater danger of the efflorescence at joints than at any other point, unless special care is taken. If the work is to be continued within 24 hours, and care is taken to scrape and remove the *laitance,* and then, before the next layer is deposited, if the scraped surface is coated with a thin cement mortar, the joint should be impervious to moisture, and no trouble with efflorescence should be experienced.

A very successful method of removing efflorescence from a concrete surface consists in applying a wash of diluted hydrochloric acid. The wash consists of 1 part acid to 5 parts water, and is applied with scrubbing brushes. Water is kept constantly played on the work, by means of a hose, to prevent the penetration of the acid. The cleaning is very satisfactory, and for plain surfaces costs about 20 cents per square yard.

Laitance. Laitance is whitish, spongy material that is washed out of the concrete when it is deposited in water. Before settling on the concrete, it gives the water a milky appearance. It is a semi-

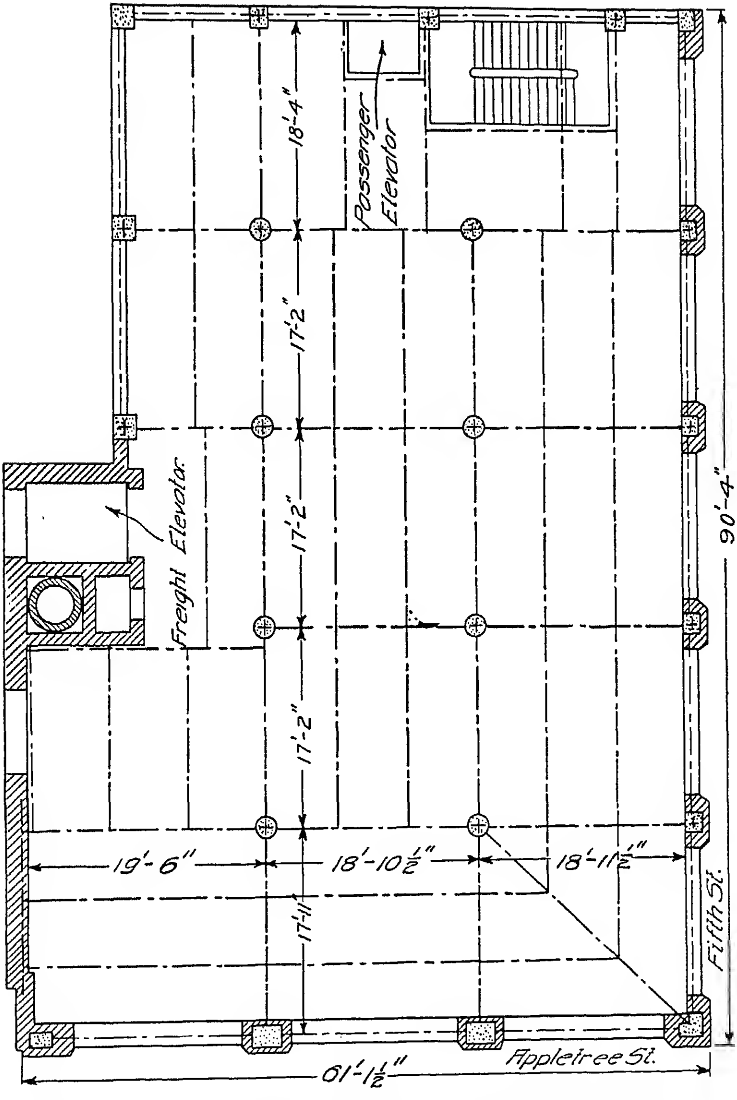


Fig. 200. Typical Structural Floor Plan of Buck Building, Philadelphia, Pennsylvania

REPRESENTATIVE EXAMPLES OF REINFORCED-CONCRETE WORK

Buck Building. Fig. 200 shows the typical structural floor-plan above the first floor, of a building constructed for J. C. Buck at Fifth and Appletree Streets, Philadelphia. The architects were Ballinger and Perrot, and the building was constructed by Cramp and Company, Philadelphia. The building has a frontage of 90 feet on Fifth Street, and a depth of 61 feet on Appletree Street, and is seven stories high. The building is constructed, structurally, of reinforced concrete, excepting the first floor and the columns in the lower floor. The floors are all designed to carry 200 pounds per square foot. The side walls are constructed of light-colored brick, and trimmed with terra cotta. The first floor, being constructed especially to suit the requirements of a chemical company that would occupy the building for several years, was planned with a view to the probable necessity of reconstructing the floor if this company should leave the building at the expiration of its lease, and hence was constructed of structural steel, since it is much easier to remodel a floor of steel than one constructed of reinforced concrete.

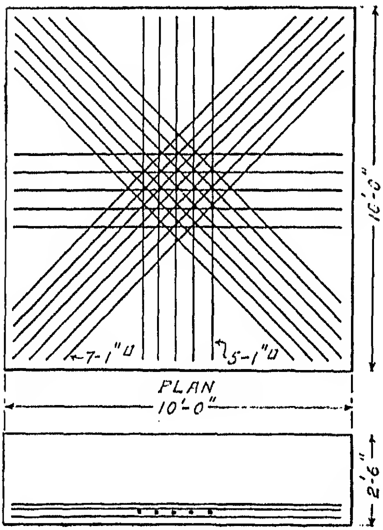


Fig. 201. Interior Column Footing for Buck Building, Philadelphia

Footings. The footings for each of the interior columns were designed as single footings. They are 10 feet square, 30 inches thick, and are reinforced as shown in Fig. 201.

Columns. The columns in the basement, first, and second floors are of structural steel, and fireproofed with concrete. The wall columns are either square or rectangular in shape; and the interior columns are round, being twenty inches in diameter. The stress

small bars placed in the column. These steel cores are provided with angle brackets to support the beams, and with spread bases to transmit the stress in the steel to the foundation. The cores are composed of angles and plates, and are riveted together in the usual manner. The columns are built in sections of a length equal to the height of two stories. This requires very little extra metal and saves the expense of half the joints required if a change of section is made at each floor.

The general outline and details of these steel cores are illustrated in Fig. 202. In the exterior columns, the steel cores are used in the basement and the first, second, and third floors, where necessary; in the interior columns, they are used also in the fourth story, and in two columns the structural steel is extended to the sixth floor line. The exterior columns above the structural steel, and also the columns in which structural steel is not required, are in general reinforced with 8 bars 1 inch square in the lower floors; and this amount of steel is gradually reduced to 4 bars 1 inch square, in the seventh story. In the interior columns, the reinforcement above the steel cores consists of 8 bars $\frac{3}{4}$ inch square, in the floor just above the structural steel; and the number of these bars is gradually reduced to 4 in the seventh floor.

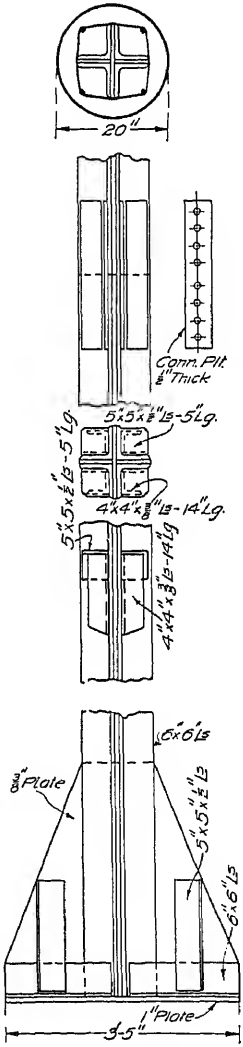


Fig. 202. Steel Column Core for Buck Building, Philadelphia, Pennsylvania

Floor Slabs. The floor slabs are 5 inches thick and reinforced with $\frac{3}{8}$-inch square bars spaced 6 inches on centers, and $\frac{5}{16}$-inch bars, spaced 24 inches on centers, the latter being placed at right angles to the former. The roof slab is designed to carry a live load of 40 pounds per square foot, and is $3\frac{1}{2}$ inches thick. The reinforcement consists of $\frac{5}{16}$-inch bars spaced 6 inches, and the same sized bars spaced 24 inches at right angles.

Floor Beams. The floor beams are, in general, 8 inches wide, and the depth below the slab is 18 inches. The amount of reinforcement in the beams varies, depending on the length of the beams. Most of the beams are reinforced with 2 bars 1 inch square, and 1 bar $1\frac{1}{8}$ inches square. The $1\frac{1}{8}$-inch bar is turned up or trussed at the ends, and the 1-inch bars are straight. The roof beams are 6- by 12-inch below the slab, and are reinforced with 2 bars $\frac{7}{8}$ inch square, except in the longest beams, in which 2 bars 1 inch square are required. A $\frac{3}{4}$-inch bar, 5 feet long, is placed in the top of all floor and roof

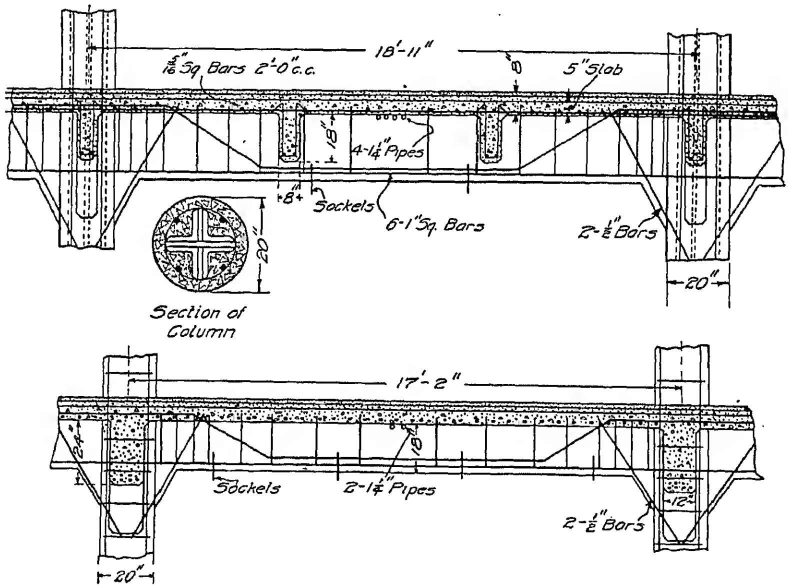


Fig. 203. Details of Beams and Girders for Buck Building, Philadelphia, Pennsylvania

beams, where they are framed into a girder. The ends of these bars are turned down. The stirrups are made of $\frac{3}{8}$-inch round bars, and are spaced as shown in the detail of the beam, Fig. 203.

Floor Girders. The floor girders are 12- by 24-inch below the slab. The span of the girders varies from about 18 feet to about 20 feet; and they are all reinforced with 6 bars 1 inch square, three of the bars being turned up at the ends. Two $\frac{3}{4}$-inch square bars are placed in the top of the girders over the supports, these bars being 5 feet long, and hooked at the ends. Bars $\frac{3}{4}$ inch square, 5 feet long,

are placed in the slab near the top, at right angles to the girders. The bars are 12 inches, center to center, and are placed over the center of the girders.

Lintels. The wall beams or lintels on the Fifth Street and Appletree Street sides of the building are shown in section in Fig. 204. They are 9 inches by 24 inches, and are reinforced with 2 bars 1 inch square. The wall girders in the side of the building opposite Appletree Street are 14 inches by 24 inches, and are reinforced with 6 bars 1 inch square.

Stairs. The stairs are constructed as shown in Fig. 205. The structural concrete slab is 6 inches thick, and is reinforced with $\frac{3}{4}$-inch bars. Safety treads $5\frac{1}{2}$ inches in width, and 12 inches shorter than the width of the stairs, are set in each step.

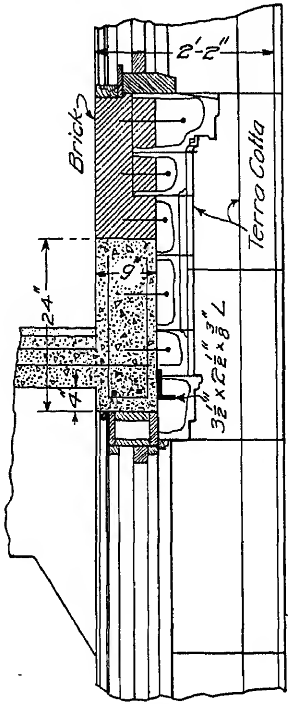


Fig. 204. Details of Wall Beams, Buck Building, Philadelphia

Concrete Mixture. The concrete for the beams, girders, slabs, and footings is a $1:2\frac{1}{2}:5$ mixture; and for the columns, a 1:2:4 mixture is required. The stone used in this concrete is trap rock. The concrete was mixed in a batch mixer, the consistency being what is commonly known as a wet mixture. Square twisted bars are used as the reinforcing steel.

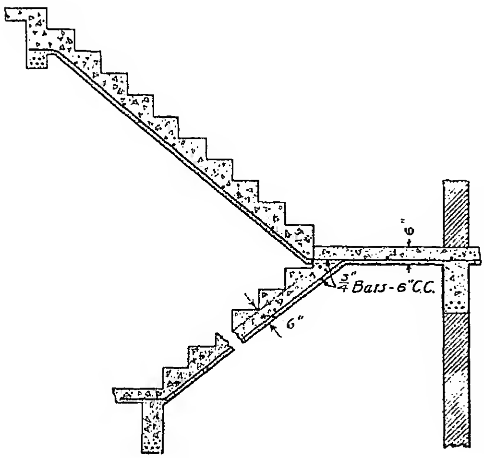


Fig. 205. Section of Stairs for Buck Building, Philadelphia

Floors. The first, second, and third floors are finished with $1\frac{1}{4}$-inch maple flooring. The stringers, 2 inches by 3

inches, are spaced 16 inches apart, and the space between the stringers is filled with cinder concrete. The other floors are finished with a one-inch coat of cement finish. A cinder fill 2 inches thick is laid on the concrete floor slab, on which was laid the cement finish. The cinder concrete consists of 1 part Portland cement, 3 parts sand, and 7 parts cinders. The cement finish is composed of 1 part Portland cement, 1 part sand, and 1 part ¼-inch crushed granite.

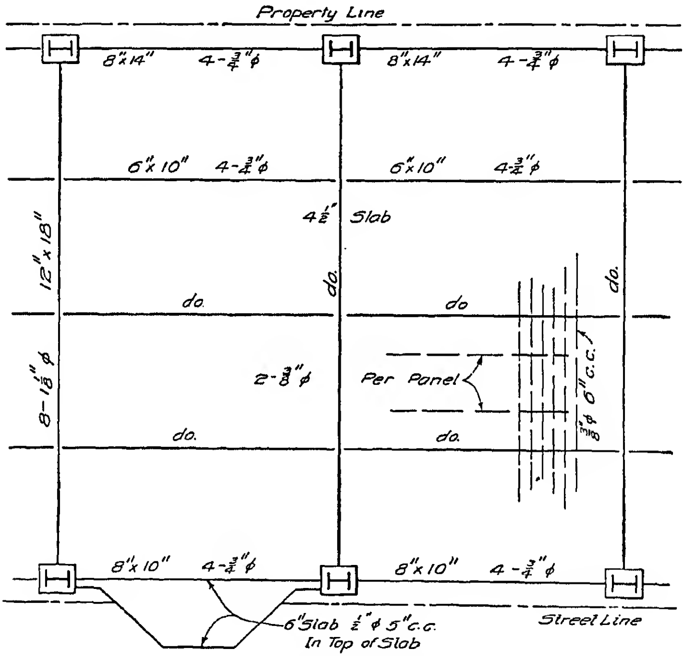


Fig. 206. Plan of Two Bays of a Floor in Allman Building, Philadelphia, Pennsylvania

Allman Building. The seven-story office building, 24 feet 9½ inches by 122 feet 2¼ inches, was constructed for Herbert D. Allman, at Seventeenth and Walnut Streets, Philadelphia. Baker and Dallett are the architects for this work. The building is constructed of reinforced concrete, except that steel-core columns are carried up to the sixth floor. Fig. 206 shows the plans of two bays of a floor, the bay windows occurring in alternate bays. The floors are designed for 120 pounds per square foot live load. The sizes of the diff-

steel is 16,000 pounds per square inch. Direct compression in the concrete is 500 pounds per square inch and the transverse stress in compression 600 pounds per square inch, while the shearing stress is 75 pounds per square inch. In designing the columns in which the steel cores occur, the radius of gyration is taken for the whole column;

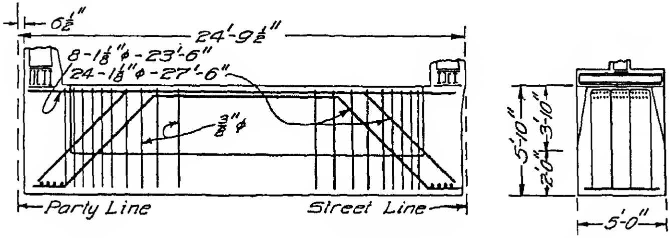


Fig. 207. Footing of Allman Building, Philadelphia, Pennsylvania

this reduces the working load to 14,000 pounds per square inch for the steel, nothing being allowed for the concrete except the increased radius of gyration. The concrete is a 1:2:4 mixture. The footings used for this building are shown in Fig. 207.

Erben-Harding Company Building. The exterior and interior of a factory building, designed and constructed by Wm. Steele and Sons Company for the Erben-Harding Company, Philadelphia, are shown in Figs. 208 and 209. This building is 100 feet by 153 feet, and is constructed structurally of reinforced concrete, except that structural steel is used in the columns. The floors and columns are designed to support safely a live load of 120 pounds per square foot.

Floor Panels. The floor panels are about 12 feet by 25 feet, the girders having a span of about 12 feet, and the beams a span of 25 feet. One intermediate beam is placed in each panel, as shown in the interior view. The girders are 12 inches wide and 20 inches deep below the slab, and are reinforced with 4 bars $1\frac{1}{16}$ inches in diameter. The beams are 12- by 18-inch, and are reinforced with 4 bars $1\frac{1}{4}$ inches in diameter. The floor slab is 4 inches thick, and is reinforced with 3-inch mesh, No. 10 gage, expanded metal.

Columns. The columns are all 18- by 18-inch; but the structural steel in the columns is designed to support the entire load on the columns. Four $\frac{3}{4}$-inch bars are placed in the columns and wrapped with expanded metal. The exterior columns are exposed to view on

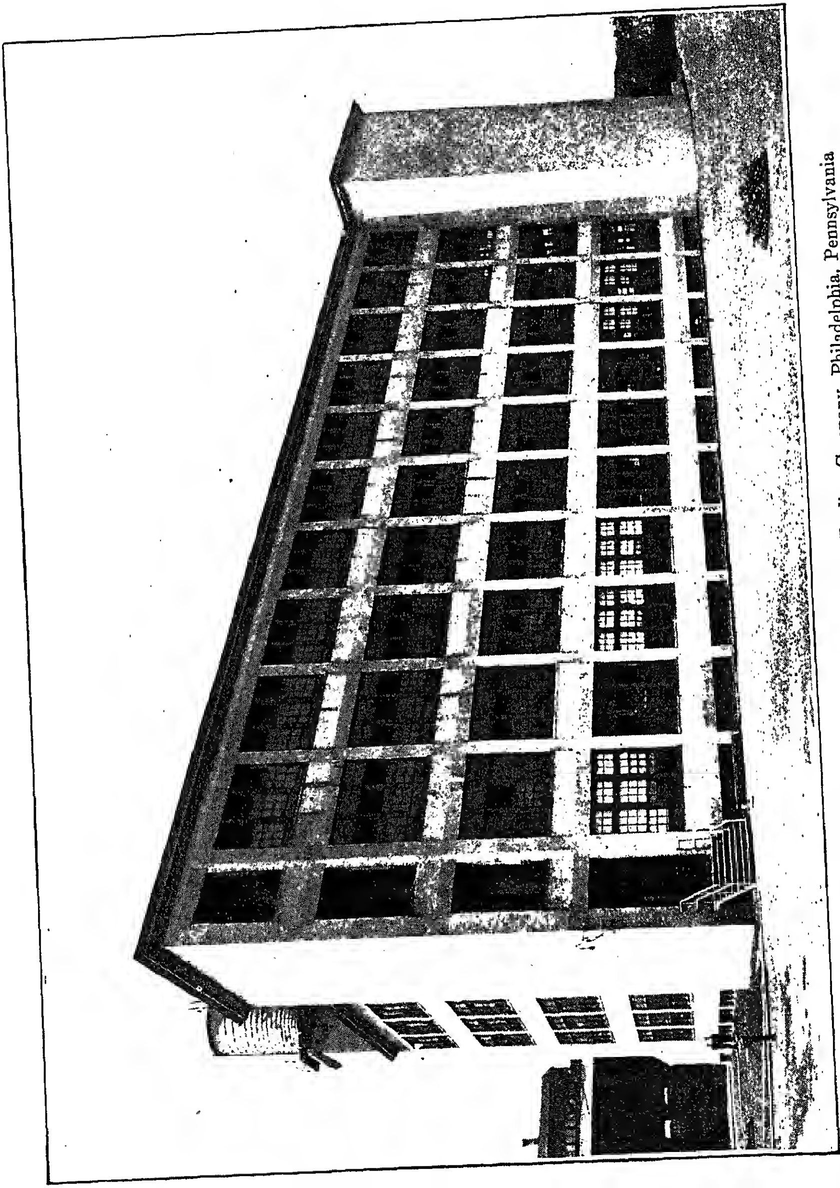


Fig. 208. Exterior of Factory Building for the Erben-Harding Company, Philadelphia, Pennsylvania
Courtesy of Wm. Steele & Sons Company, Philadelphia, Pennsylvania

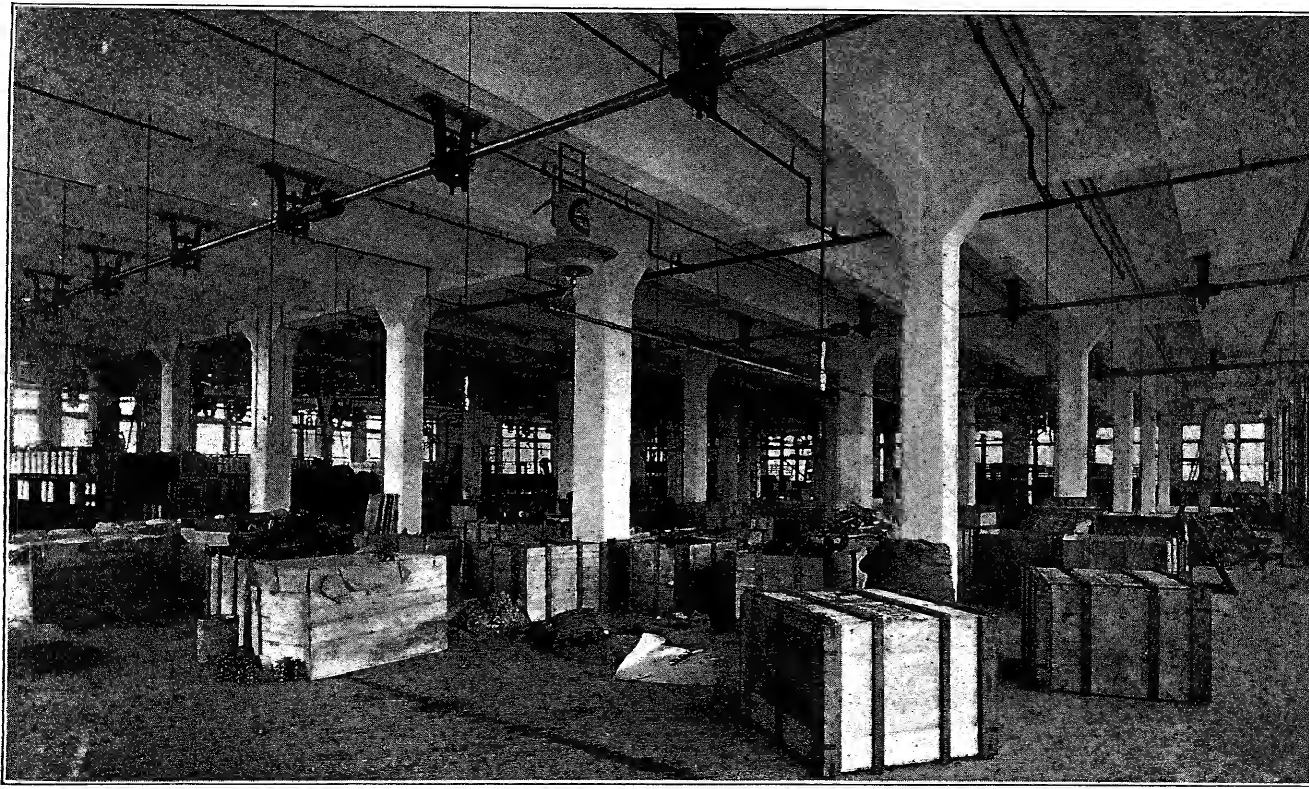


Fig. 209. Interior of Factory Building for the Erben-Harding Company, Philadelphia, Pennsylvania
Courtesy of Wm. Steele & Sons Company, Philadelphia, Pennsylvania

between the wall columns is filled by triple windows. The wall beams are constructed flush with the exterior surface of the wall columns, as shown in Fig. 208. The space between the bottom of the windows and the wall beams is filled with white brick. The two fire towers, located at the corners of the building, are also constructed of white brick.

Floor Finish. The floor finish of this building is somewhat unusual. Sills 2 by 4 inches are laid on the structural floor slab of concrete, and the space between these sills is filled with cinder concrete. On these sills is laid a covering of 2-inch tongued-and-grooved plank; and on these planks is laid a floor of $\frac{7}{8}$-inch maple, the latter being laid perpendicular to the 2-inch plank.

Swarthmore Shop Building. In constructing the shop building at Swarthmore College, Swarthmore, Pennsylvania, concrete blocks

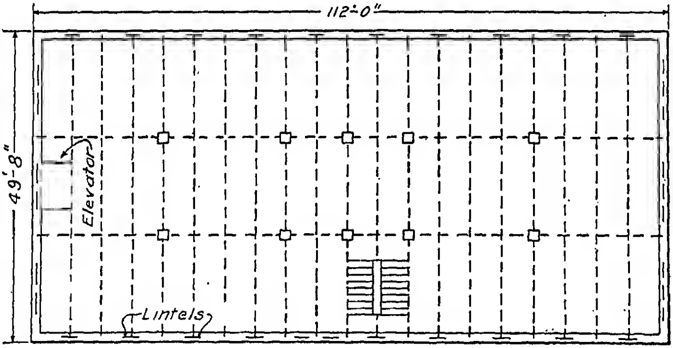
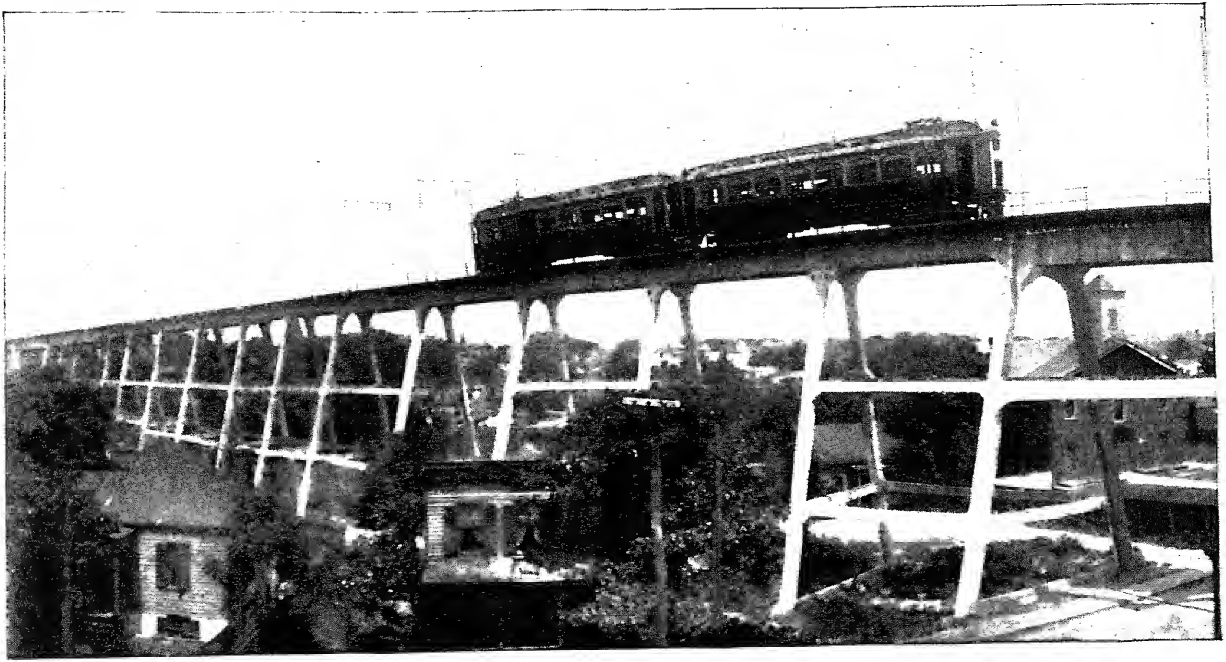


Fig. 210. Plan of Shop Building, Swarthmore College, Swarthmore, Pennsylvania

were used for the side walls, and the floors were constructed of reinforced concrete. This building is 49 feet 8 inches by 112 feet, and is 3 stories high. The floors are designed to carry a live load of 150 pounds per square foot. A factor of safety of 4 was used in all the reinforced-concrete construction.

The columns are located as shown in Fig. 210. The span of the girders is 20 feet, except for the three middle bays, in which the span is only 10 feet. The 20-foot girders are 14 inches wide, and the depth below the slab is 23 inches. The reinforcement consists of 8 bars



CONCRETE VIADUCT OF RICHMOND & CHESAPEAKE BAY RAILWAY

Interesting Example of Fireproof Viaduct Construction. Kahn System of Reinforced Concrete as Furnished by the Trussed Concrete Steel Company, Detroit, Used in This Construction

depth 12 inches below the slab; and the reinforcement consists of 5 bars $\frac{5}{8}$ inch square. The slab is 4 inches thick, including the top coat

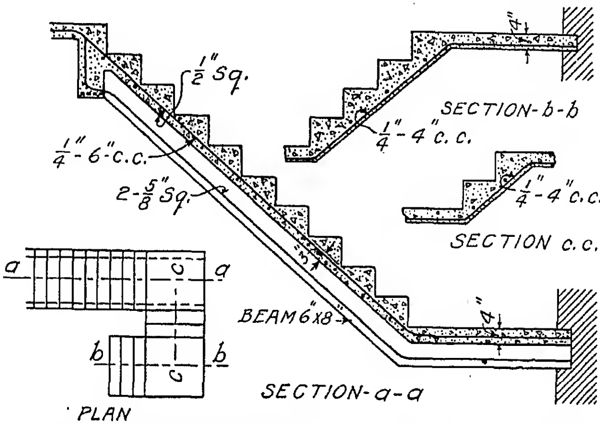


Fig. 211. Stairway Details in Shop Building, Swarthmore College, Swarthmore, Pennsylvania

of 1 inch, which is composed of 1 part Portland cement and 1 part sand. This finishing coat was put on before the other concrete had set, and was figured as part of the structural slab. The slab reinforcement consists of $\frac{1}{4}$-inch bars spaced 4 inches on centers, and $\frac{1}{4}$-inch bars spaced 24 inches at right angles to the bars spaced 4 inches. The columns range in size from 10- by 10-inch to 18- by 18-inch, and are reinforced by placing a bar in each corner of the column, which bars are tied together by $\frac{1}{4}$-inch bars spaced 12 inches. The amount of this steel is about one per cent of the total area of the column.

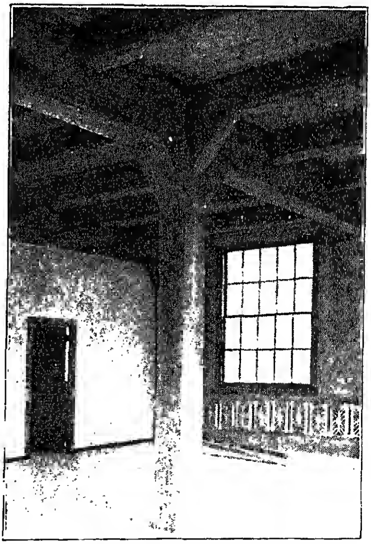


Fig. 212. Floor Construction in Shop Building of Swarthmore (Pa.) College Showing Connection of Girder Beams with Column

Fig. 211 shows the plans of the stairway. The lintels were

to the proper height. The size of the lintels varies on the different floors to conform with the architectural features of the building. The width of the lintels is made the same as the thickness of the walls, and therefore both sides of the lintels are exposed to view. They are reinforced with 3 bars ½ inch square.

The concrete was composed of 1 part Portland cement, 3 parts sand, and 5 parts stone. The stone was graded in size from ¼ inch to 1 inch. Johnson corrugated bars were used as the reinforcing steel. A panel, 16 by 20 feet, of one of the floors, was tested by placing a load of 300 pounds per square foot over this area. The deflection was so slight that it could not be conveniently measured. In Fig. 212 is given a view of the under side of a floor, showing the connection of the girder and beams with the column.

Tile and Joist System. The tile and joist system of constructing fireproof floors is found economical for a certain class of work. It

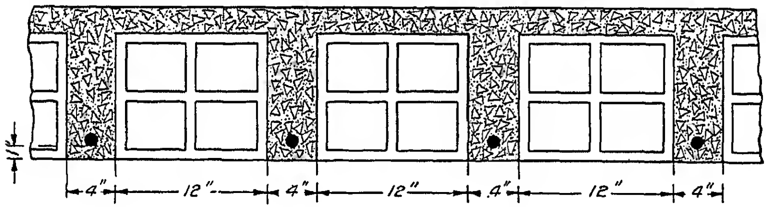


Fig. 213. Tile and Joist Construction

is probably used for apartment houses oftener than anywhere else. The advantage secured by this construction is that a flat ceiling is secured. The structural frame of the building may be either steel or reinforced concrete. The columns are connected by girders and the space between the girders is filled in with tile and joists. When reinforced-concrete girders are used between the columns, a slab of concrete of sufficient width and thickness to take the compression must be constructed.

Fig. 213 shows a section of a tile and joist floor. The terra cotta tile is always 12 inches in width and from 4 inches to 15 inches in depth. The tile is simply a filler between the joists and is so much dead weight to be carried by the joists. The joists are usually 4 inches in width and are designed as T-beams. The slab is usually

Fig. 214. Interior of H. T. Heinz Company Warehouse, Showing Example of Flat-Slab Ceiling Construction
Courtesy of The Condron Company, Chicago

of one bar of sufficient area for the tensile stress. The slab should be reinforced with $\frac{1}{4}$-inch bars, 24 inches center to center each way.

Heinz Warehouse. A good example of a ceiling of the flat-slab system is given in Fig. 214. This shows an interior view of a warehouse designed by the Condron Company of Chicago, Illinois, for the H. T. Heinz Company, Chicago. The panels are 18 feet 6 inches square and are designed for a live load of 300 pounds per square foot.

Steel Cores. It is often necessary, in reinforced-concrete buildings, to construct columns of some other material than concrete on account of the large space that would be occupied by the columns. In such cases steel-core columns are often used. Fig. 215 shows two types of the steel cores. Type *a* is used for round columns and the steel consists of four angles, but, when necessary, plates are inserted between the angles to make up the full section. Type *b* is used for square columns. In figuring the strength of these columns, the Bureau of Building Inspection of Philadelphia will permit the steel to be figured as having a radius of gyration equal to that of the concrete section, which for ordinary story heights makes the permissible loading about 14,000 pounds per square inch, but additional loading is not permitted on the concrete. The steel must be surrounded by at least 2 inches of concrete, in which there must be placed 4 small vertical bars, usually $\frac{3}{4}$ inch, banded by $\frac{1}{4}$-inch bars, 12 inches on centers. The loads are transmitted from the beams and girders to the steel by means of large steel brackets which are riveted to the columns. The work is riveted up in the usual manner for structural steel.

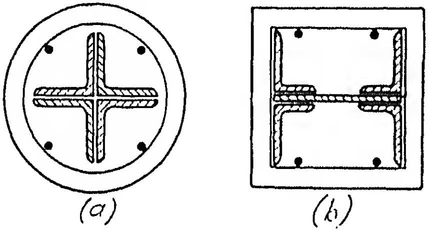


Fig. 215. Typical Sections of Steel Core Columns

The McNulty Building.* The columns used in the construction of the McNulty Building, New York City, are a very interesting feature in this building. The building is 50 feet by 96 feet, and is 10 stories high. The plan of all the floors is the same. A single row of interior columns is placed in the center of the building, about 22

The columns are of the hooped type, and are designed from the formula approved by the building laws of New York City. The formula used was

$$P = 1{,}600r^2 + (160{,}000\, A_h \div P) \times r + 6{,}000\, A_s$$

in which P is the total working load, r is the radius of the helix, A_s is the total area of the vertical steel, A_h is the sectional area of the hooping wire, and P is the pitch of the helix.

The interior columns are cylindrical in form, except those supporting the roof, which are 12- by 12-inch and are reinforced with 4 bars $\frac{3}{4}$ inch in diameter. In all the other stories except the ninth, they are 27 inches in diameter. Below the fifth floor the reinforcement in each of these columns consists of 2-inch round vertical bars, ranging in number from 7 in the fifth floor to 30 in the basement, and banded by a 24-inch helix of $\frac{1}{2}$-inch wire, with a pitch of $1\frac{1}{2}$ inches. The vertical bars were omitted between the sixth and tenth floors; and the diameter of the helix was gradually decreased, while the pitch was increased. In the ninth floor the diameter was reduced to 21 inches.

The wall columns are, in general, 26 by 30 inches, and support loads from 48,000 pounds in the tenth floor to 719,750 pounds in the basement. In the sixth story, the reinforcement in these columns consists of 3 round, vertical bars 2 inches in diameter; and in each of the floors below, the number of bars in these columns was increased, there being 24 in the basement columns. These are spirally wound with $\frac{5}{16}$-inch steel wire forming a helix 23 inches in diameter, with a pitch of $2\frac{1}{2}$ inches. Above the seventh floor, the columns are reinforced with 4 bars $\frac{3}{4}$ inch in diameter, and tied together by $\frac{5}{16}$-inch wire spaced 18 inches apart. The columns rest on cast-iron shoes, which are bedded on solid rock about $2\frac{1}{2}$ feet below the basement floor.

The main-floor girders extend transversely across the building, and have a clear span of 21 feet. The floor beams are spaced about 6 feet apart, and have a span of about 20 feet 6 inches. The sides of the beams slope, the width at the bottom being two inches less than the width at the under surface of the slab. The reinforcement consists of plain round bars. The bars for the girders and beams were bent and made into a truss—the *Unit System*—at the shops of the

The stirrups were hot-shrunk on the longitudinal bars. The helixes for the columns were wound and attached to some of the vertical rods at the shop, to preserve the pitch. The vertical rods in each column project 6 inches above the floor line, and are connected to the bar placed on it by a piece of pipe 12 inches long.

The concrete was a 1:2:4 mixture. Giant Portland cement was used, and $\frac{3}{4}$-inch trap rock.

The McGraw Building. The McGraw Building, New York City, is a good example of a reinforced-concrete building. The building has a frontage of 126 feet and a depth of 90 feet, and is 11

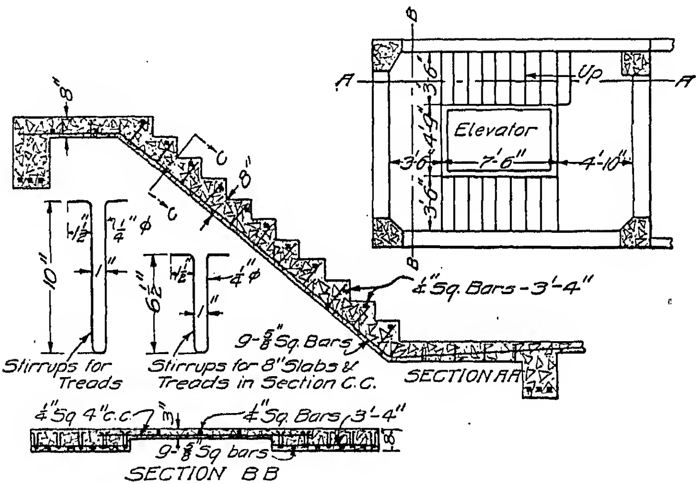


Fig. 216. Stair Details for the Fridenberg Building

stories in height. The height of the roof is about 150 feet above the street level. The building was designed to resist the vibration of heavy printing machinery. The first and second floors were designed for a live load of 250 pounds per square foot; for the third floor, 150 pounds per square foot; for the fourth floor and all floors above the fourth floor, 125 pounds per square foot.

All beams and girders were designed as continuous beams, even where supported on the outside beams. There is twice as much steel over the supports as in the center of the spans. The Building Code of New York City requires that the moment for continuous beams be taken as $\frac{Wl}{}$ at the center of the span, and as $\frac{Wl}{}$ over the

support. These values are more than twice the theoretical value as computed for continuous beams.

One very interesting feature of this building is that it was constructed during the winter. The first concrete was laid during September, and the concrete work was completed in April. During freezing weather, the windows of the floors below the floor that was being constructed were closed with canvas; and salamanders (open stoves) were distributed over the completed floor, and kept in constant operation. Coke was used as the fuel for the salamanders. The concrete was mixed with hot water, and the sand and the stone were also heated. After two or three stories had been erected, and the construction force was fully organized, a floor was completed in about 12 days. Three complete sets of forms were provided and used. They were usually left in place nearly three weeks.

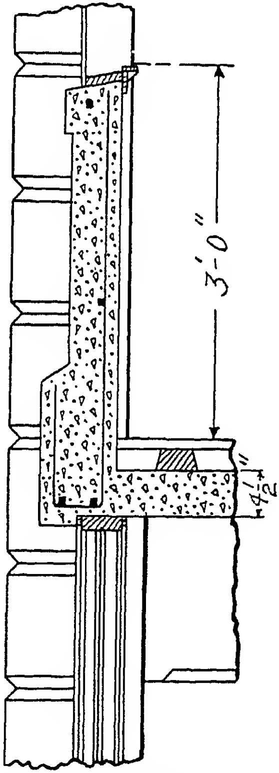


Fig. 217. Details of Special Type of Lintel

Fridenberg Building. In Fig. 216 are shown the plans of stairs constructed in the Fridenberg building at 908 Chestnut Street, Philadelphia. This building is 24 feet by 60 feet, and is seven stories high. Structurally, the building is constructed of reinforced concrete. The stair and elevator tower is located in the rear of the main building.

The plans of the stairs are interesting on account of the long-span (about 16 feet) slab construction. The stairs are designed to carry safely a live load of 100 pounds per square foot; and in the theoretical calculations the slab was treated as a flat slab with a clear span of 16 feet. The shear bars are made and spaced as shown in the details. The calculations showed a low shearing value in the concrete, but stirrups were used to secure a good bond between

The concrete was a 1:2:4 mixture, and was mixed wet. The reinforcing steel consisted of square deformed bars, except the stirrups, which were made of ¼-inch plain round steel.

Special Type of Lintel. An interesting feature of a large reinforced-concrete building constructed for the General Electric Company at Fort Wayne, Indiana, is the design of the lintels. As shown in Fig. 217, the bottom of the lintel is at the same elevation as the bottom of the slab. The total space between the columns is filled with double windows; and the space from the top of these windows to the bottom of those above, is filled with a beam which also serves as a wall.

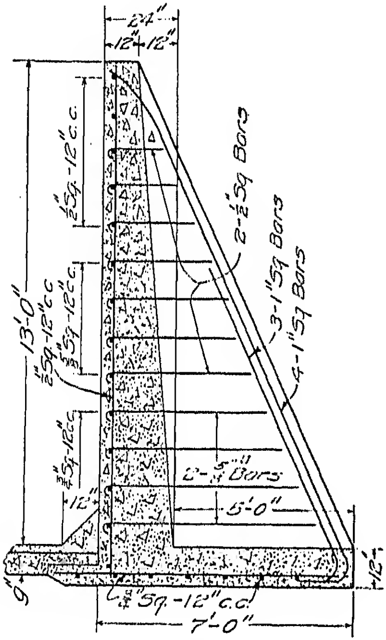


Fig. 218. Typical Section of Water-Basin Wall

Water-Basin and Circular Tanks. Figs. 218 and 219 illustrate sections of the walls of the pure water basin and the 50-foot circular tanks which have been partly described in Part I, page 69, under the heading of "Waterproofing".

The pure water basin is 100 feet by 200 feet, and 14 feet deep, giving a capacity of over 1,500,000 gallons. The counterforts are spaced 12 feet 6 inches, center to center, and are 12 inches thick, except every fourth one, which was made 18 inches thick. The 18-inch counterforts were constructed as two counterforts each 9 inches thick, as the vertical joints in the walls were made at this point; that is, the concrete between the centers of two of the 18-inch counterforts was placed in one day. On the two ends and one side of the basin the counterforts were constructed on the exterior of the basin to support about 10 feet of earth. But on one side it would have been necessary to remove rock 6 to 8 feet in thickness to make room for the counterforts, had they been constructed on the exterior

If both faces of the vertical wall had been reinforced, the same as the one shown, then the wall would have been able to resist an outward or an inward pressure, and the "piers" would act as counterforts or buttresses, depending on whether they were in tension or in compression.

The concrete used consisted of 1 part Portland cement, 3 parts sand, and 5 parts crushed stone. The stone was graded in size from $\frac{1}{4}$ inch as the minimum to $\frac{3}{4}$ inch as the maximum size. Square-sectioned deformed bars were used as the steel reinforcement. The forms were constructed in units so that they could be put up and taken down quickly.

The size and spacing of the bars in the walls of the circular tanks are shown in Fig. 219. The framework of the forms to which the lagging was fastened was cut to the desired curve at a planing mill. This framing was cut from 2- by 12-inch lumber. The lagging was $\frac{7}{8}$ inch thick, and surfaced on one side.

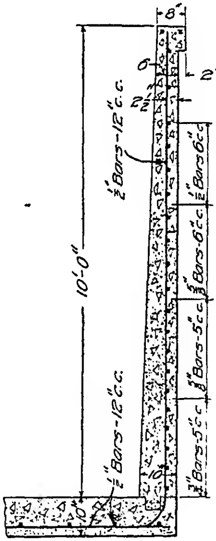


Fig. 219. Typical Section of Tank

Main Intercepting Sewer. In the development of sewage purification work at Waterbury, Connecticut, the construction of a main intercepting sewer was a necessity. This sewer is three miles long. It is of horseshoe shape, 4 feet 6 inches by 4 feet 5 inches, and is constructed of reinforced concrete. The details are illustrated in Fig. 220.

The trench excavations were principally through water-bearing gravel, the gravel ranging from coarse to fine. Some rock was encountered in the trench excavations. It was a granite gneiss of irregular fracture, and cost, with labor at $17\frac{1}{2}$ cents per hour, about \$2.00 per cubic yard to remove it. Much of the trench work varied in depth from 20 to 26 feet. Owing to the varying conditions, it was necessary to vary the sewer section somewhat. Frequently, the footing course was extended. However, the section shown in the figure is the normal section.

The concrete was mixed very wet, and poured into practically

cement to $7\frac{1}{2}$ parts of aggregate, graded to secure a dense concrete. Care was used in placing the concrete, and very smooth surfaces were secured. Plastering of the surfaces was avoided. Any voids were grouted or pointed, and smoothed with a wood float. Expanded metal and square-twisted bars were used in different parts of the work. In Fig. 220, the size and spacing of the bars are shown. The bars were bent to their required shape before they were lowered into the excavation.

The forms in general were constructed as shown in the figure. The inverted section was built as the first operation; and after the

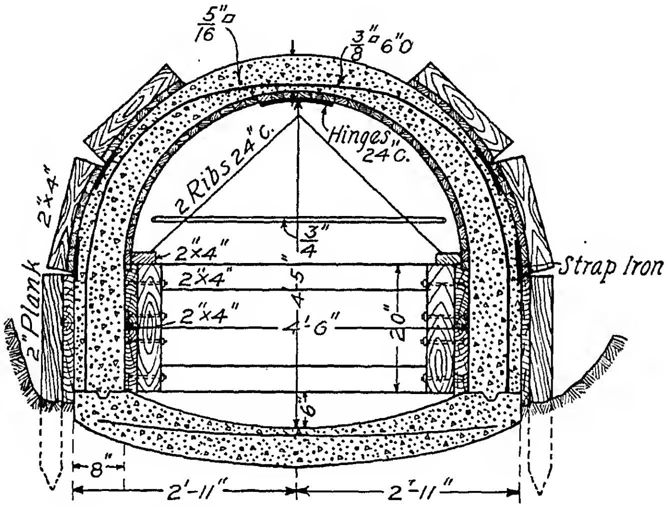


Fig. 220. Section of Intercepting Sewer at Waterbury, Connecticut

surface was thoroughly troweled, the section was allowed to set 36 to 48 hours before the concreting of the arch section was begun. The lagging was $\frac{7}{8}$ inch thick, with tongue-and-groove radial joints, and toenailed to the 2-inch plank ribs. The exterior curve was planed and scraped to a true surface. The vertical sides of the inner form are readily removable, and the semicircular arch above is hinged at the soffit and is collapsible. The first cost of these forms has averaged \$18.00 for 10 feet of length; and the cost of the forms per foot of sewer built, including first cost and maintenance, averaged 10 cents. Petrolene, a crude petroleum, was found very effective in preventing

Cost records kept under the several contracts and assembled into a composite form show what is considered to be the normal cost of this section, under the local conditions. Common labor averaged $17\frac{1}{2}$ cents, sub-foremen 30 cents, and general foremen 50 cents per hour.

Normal Cost per Lineal Foot of 53- by 54-Inch Reinforced-Concrete Sewer

Steel reinforcement, $17\frac{1}{8}$ lb.	$0.43
Making and placing reinforcement cages	.14
Wood interior forms, cost, maintenance, and depreciation	.12
Wood exterior forms, cost, maintenance, and depreciation	.05
Operation of forms	.16
Coating oil	.01
Mixing concrete	.30
Placing concrete	.27
Screeding and finishing invert	.08
Storage, handling, and cartage of cement	.08
0.482 bbl. cement at $1.53	.74
0.17 cu. yd. sand at $0.50	.09
0.435 cu. yd. broken stone at $1.10	.47
Finishing interior surface	.01
Sprinkling and wetting completed work	.02
Total cost per lineal foot	$2.97

This is equivalent to a cost of $9.02 per cubic yard.

Bronx Sewer, New York. In Fig. 221 is shown a section of one of the branch sewers constructed in the Borough of the Bronx, New York City. A large part of this sewer is located in a salt marsh where water and unstable soil made construction work very difficult. The general elevation of the marsh is 1.5 feet above mean high water. In constructing this sewer in the marsh, it was necessary to construct a pile foundation to support the sewer. The foundation was capped with reinforced concrete; and then the sewer, as shown in the section, was constructed on the pile foundation. The concrete for this work is composed of 1

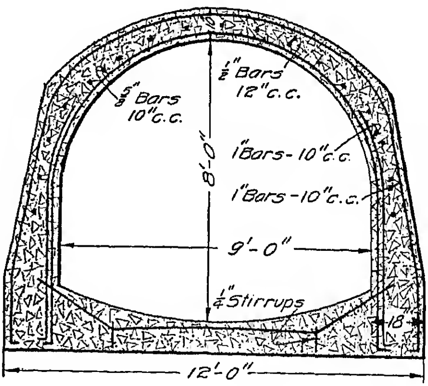


Fig. 221. Section of Bronx Sewer, New York City

for the reinforcement in the work.

Girder Bridge. The reinforced-concrete bridge shown in Fig. 222 was constructed near Allentown, Pennsylvania. This type of bridge has been found to be economical for short spans. Worn-out wood and steel highway bridges are in general being replaced with reinforced-concrete bridges, and usually at a cost less than that of a steel bridge of the same strength. Steel bridges should be painted every year; and plank floors, as commonly used in highway bridges, require almost constant attention, and must be entirely renewed several times during the life of a bridge. A reinforced-concrete

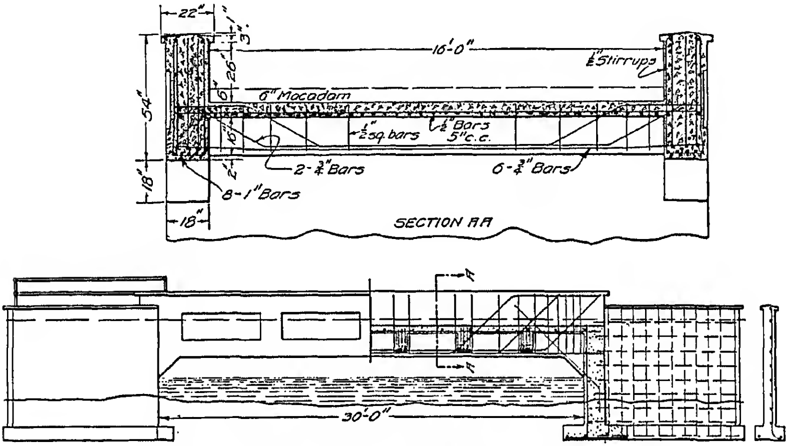


Fig. 222. Details of Girder Bridge near Allentown, Pennsylvania

bridge, however, is entirely free of these expenses, and its life should at least be equal to that of a stone arch. From an architectural standpoint, a well-finished concrete bridge compares very favorably with a cut-stone arch.

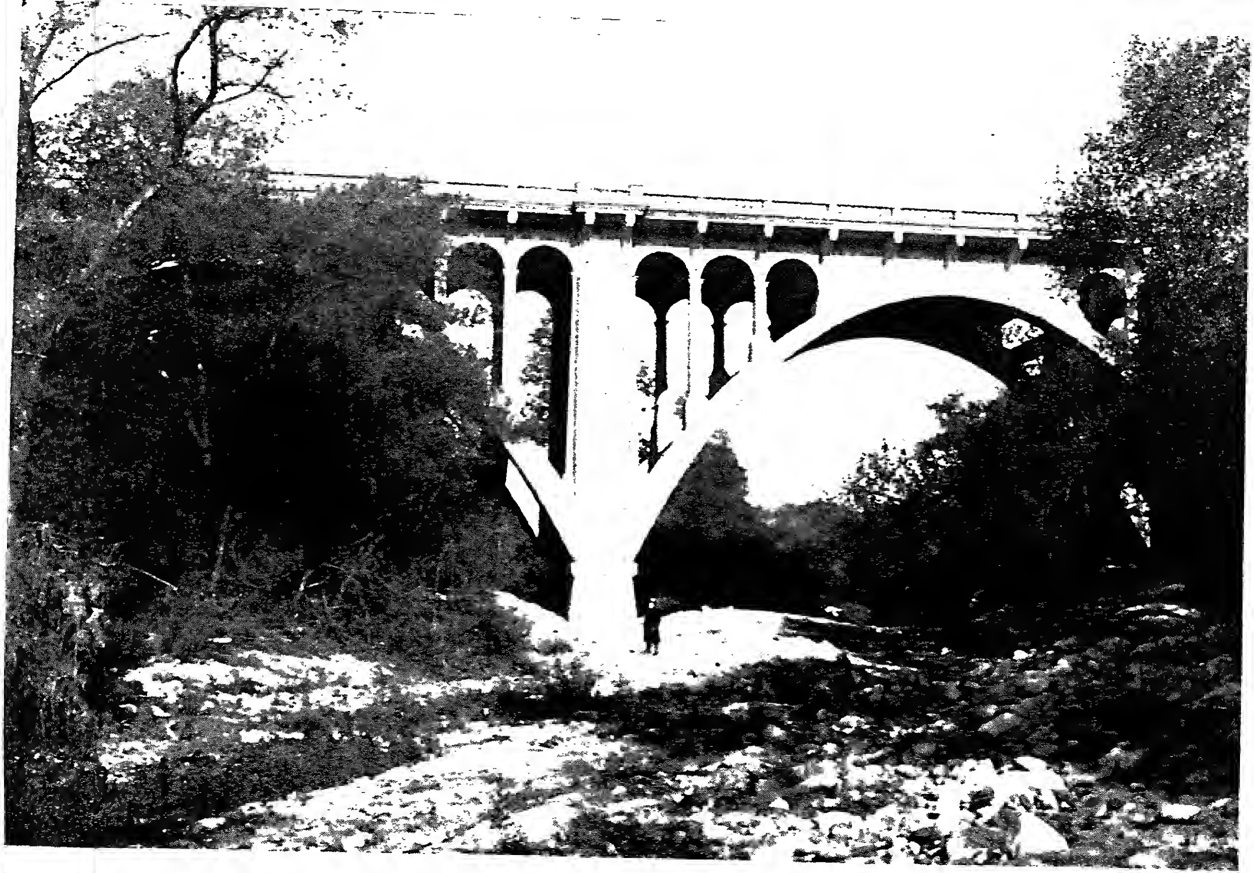
The bridge shown in Fig. 222 is 16 feet wide, and has a clear span of 30 feet. It is designed to carry a uniformly distributed load of 150 pounds per square foot, or a steel road roller weighing 15 tons, the road roller having the following dimensions: The width of the front roller is 4 feet, and of each rear roller, 20 inches; the distance apart of the two rear rollers is 5 feet, center to center; and the distance between front and rear rollers is 11 feet, center to center;

the weight on the front roller is 6 tons, with 4.5 tons weight on each of the rear rollers.

In designing this bridge, the slab was designed to carry a live load of 4.5 tons on a width of 20 inches, when placed at the middle of the span, together with the dead load consisting of the weight of the macadam and the slab. The load considered in designing the cross-beams consisted of the dead load—weight of the macadam, slab, and beam—and a live load of 6 tons placed at the center of the span of the beam, which was designed as a T-beam. In designing each of the longitudinal girders, the live load was taken as a uniformly distributed load of 150 pounds per square foot over one-half of the floor area of the bridge. The live load was increased 20 per cent over the live load given above, to allow for impact.

In a bridge of this type, longitudinal girders act as a parapet, as well as the main members of the bridge. The concrete for this work was composed of 1 part Portland cement, 2 parts sand, and 4 parts 1-inch stone. Corrugated bars were used as the reinforcing steel.

When there is sufficient headroom, all the beams can be constructed in the longitudinal direction of the bridge, and are under the slab. The parapet may be constructed of concrete; or a cheaper method is to construct a handrailing with $1\frac{1}{2}$-inch or 2-inch pipe.



ARROYO TERRACE BRIDGE, PASADENA, CALIFORNIA

An excellent example of an artistic reinforced concrete arch. The bridge consists of two spans and cost $50,000.

MASONRY AND REINFORCED CONCRETE

PART V

CONCRETE ARCH DESIGN AND CONSTRUCTION

Definitions of Terms Pertaining to Arch Masonry. The following are definitions of technical terms frequently used in connection with the subject of arch masonry (see Fig. 223):

Abutment. An abutment is the masonry which supports an arch at either end, and which is so designed that it can resist the lateral thrust of the arch.

Arch Sheeting. Arch sheeting is that portion of an arch which lies between the ring stones.

Backing. Backing is the masonry which is placed outside of or above the extrados, with the sole purpose of furnishing additional weight on that portion of the arch; it is always made of an inferior quality of masonry and with the joints approximately horizontal.

Coursing Joint. A coursing joint is a mortar joint which runs continuously from one face of the arch to the other.

Crown. The crown is the vertex or highest part of an arch ring.

Extrados. The extrados is the upper, or outer, surface of the voussoirs which compose the arch ring.

Haunch. That portion of an arch which is between the crown and the skewback is called the haunch; although there is no definite limitation, the term applies, generally, to that portion of the arch ring which is approximately halfway between the crown and the skewback.

Heading Joint. A heading joint is a joint between two consecutive stones in the same string course. In order that the arch shall be properly bonded together, such joints are purposely made *not* continuous.

Intrados. The intrados is the inner or lower surface of an arch. The term is frequently restricted to the line which is the intersection of the inner surface by a plane that is perpendicular to the axis of the arch.

Keystone. The keystone is the voussoir which is placed at the crown of an arch.

Parapet. The wall which is usually built above the spandrel walls and above the level of the roadway is termed the parapet.

Ring Stones. Ring stones are the voussoirs which form the arch ring at each end of the arch.

Rise. The rise is the vertical height of the bottom of the keystone above the plane of the skewbacks.

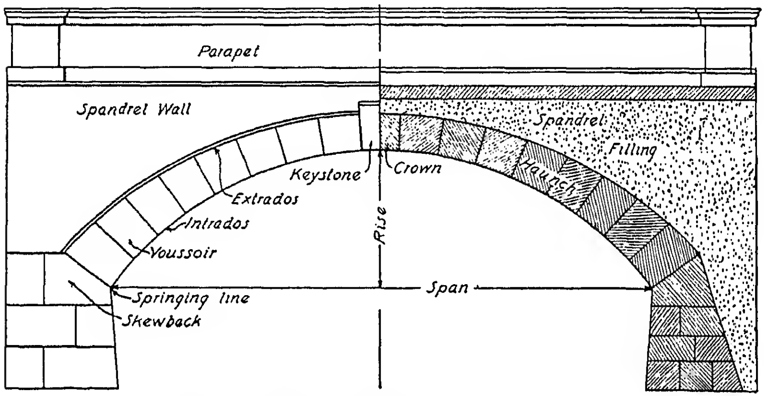


Fig. 223. Diagram Showing Parts of a Typical Arch

Skewback. Skewback is the term applied to the top course of stones on the abutments. The upper surfaces of the stones are cut at such an angle that the surfaces are approximately perpendicular to the direction of the thrust of the arch.

Soffit. The inner or lower surface of an arch is known as the soffit.

Span. The span is the perpendicular distance between the two springing lines of an arch.

Spandrel. The space between the extrados of an arch and the roadway is designated as the spandrel. The walls above the ring stones at the ends of the arch are called *spandrel walls.* The material deposited between the spandrel walls and in this spandrel space

Springer. Springer is, loosely, the point from which an arch seems to spring; or specifically, the first arch stone above a skew-back.

Springing Line. The springing line is the upper (and inner) edge of the line of skewbacks on an abutment.

String Course. A string course is a course of voussoirs of the same width—perpendicular to the axis of the arch—which extends from one arch face to the other.

Voussoirs. Voussoirs are the separate stones forming an arch ring.

Classification of Arches. Arches are variously described according to the shape of the intrados, and also according to the form of the soffit:

Basket-Handle Arch. A basket-handle arch is one whose intrados consists of a series of circular arcs tangent to each other. They are usually *three-centered* or *five-centered.*

Catenarian Arch. A catenarian arch is an arch whose intrados is a mathematical curve known as the *catenary.* This is the natural curve assumed by a chain which is hung loosely from two points.

Circular Arch. Circular arches are those in which the intrados is the arc of a circle.

Elliptical Arch. An elliptical arch is an arch whose intrados is a portion of an ellipse.

Hydrostatic Arch. A hydrostatic arch is one whose intrados is of such a form that the equilibrium of the arch is dependent upon such a loading as would be made by water.

Pointed Arch. A pointed arch is one whose intrados consists of two similar curves which meet at a point at the top of the arch.

Relieving Arch. An arch which is built above a lintel, and which relieves the lintel of the greater portion of its load, is called a relieving arch.

Right Arch. A right arch is an arch whose soffit is a cylinder, and whose ends are perpendicular to the axis of the arch.

Segmental Arch. A segmental arch is one whose intrados is a circular arc which is less than a semicircle.

Semicircular Arch. A semicircular arch is one whose intrados

Skew Arch. A skew arch is an arch whose soffit may or may not be cylindrical, but whose ends are not perpendicular to the axis of the arch. They are also called *oblique arches.*

THEORY OF ARCHES

General Statement. The mechanics of the arch are almost invariably solved by a graphical method, or by a combination of the graphical method with numerical calculations. This is done, not only because it simplifies the work, but also because, although the accuracy of the graphical method is somewhat limited, yet, with careful work, it may easily be made even more accurate than is necessary, considering the uncertainty as to the true ultimate strength of the masonry used. The development of this graphical method must necessarily follow the same lines as in Statics. It is here assumed that the student has a knowledge of Statics, and that he already understands the graphical method of representing the magnitude, direction, and line of application of a force. Several of the theorems or general laws regarding the composition and resolution of forces, will be briefly reviewed as a preliminary to the proof of those laws of graphical statics which are especially applied in computing the stresses in an arch.

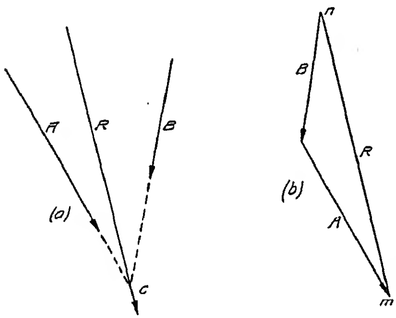


Fig. 224. Diagram of Resultant of Two Forces

Resultant of Two Non-Parallel Forces. The resultant of two forces, A and B, which are not parallel, whose lines of action are as shown in Fig. 224-a, and which are measured by the *lengths* of the lines A and B, Fig. 224-b, is readily found by producing the lines of action to their intersection at c. The two known forces are drawn in Fig. 224-b, so that their direction is parallel to the known directions of the forces, and so that the point of one force is at the butt end of the other. Then the line R joining the points m and n, Fig. 224-b, gives the direction of the resultant; and a line through c parallel to that direction gives the actual line of that resultant. The line mn

a *closed figure.* If an arrow is marked on R so that it points *upward,* the arrows on the forces would run continuously around the figure. If R were acting upward, it would represent the force which would just hold A and B in equilibrium; pointing *downward,* it is the resultant or combined effect of the two forces. We may thus define the *resultant* of two—or more—forces as the force which is the *equal* and *opposite* of that force which will just hold that combination of forces in equilibrium.

Resultant of Three or More Forces. This may be solved by an extension of the method previously given as shown in Fig. 224. The resultant of B and C, Fig. 225, is R'; and this is readily combined with A, giving R'' as the resultant of all three forces. The same principle may be extended to any number of non-parallel forces acting in a plane. The resultant of four non-parallel forces is best determined by finding, first, the resultant of each pair of the forces taken two and two. Then the resultant of the two resultant forces is found, just as if each resultant were a single force.

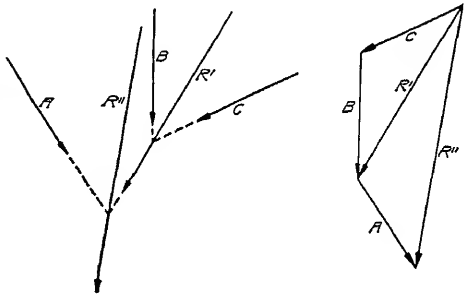


Fig. 225. Diagram Showing Resultant of Three Forces

Resultant of Two or More Parallel Forces. When the forces are all parallel, the *direction* of the resultant is parallel to the component forces; the *amount* is equal to the *sum* of the component forces; but the *line of action* of the resultant is not determinable as in the above cases, since the forces do not intersect. It is a principle of Statics which is easily appreciated, that it does not alter the statics of any combination of forces to assume that two *equal* and *opposite* forces are applied along any line of action. From Fig. 226-b, we see that the forces F and G will hold A in equilibrium; that G and H will hold B in equilibrium; and that H and K will hold C in equilibrium. But the force G required to hold A in equilibrium is the equal and opposite of the force G required to hold B in equilibrium; and similarly the force H for B is equal and opposite to the force H for C. We thus find that the forces A, B, and C can be held in equi-

equal and opposite forces H, and the unbalanced force K. The result, therefore, is that A, B, and C are held in equilibrium by two forces F and K. The resultant R is the *sum* of A, B, and C; therefore the combined-load line represents the resultant R. external lines of Fig. 226-b show that F, K, and R form a closed e with the arrows running continuously around the figure; and F and K are two forces which hold R, the resultant of A, B, and n equilibrium. By producing the lines representing the forces nd K in Fig. 226-a until they intersect at x, we may draw a ical line through it which gives the desired line of action of R. is in accordance with the principles given in the previous le.

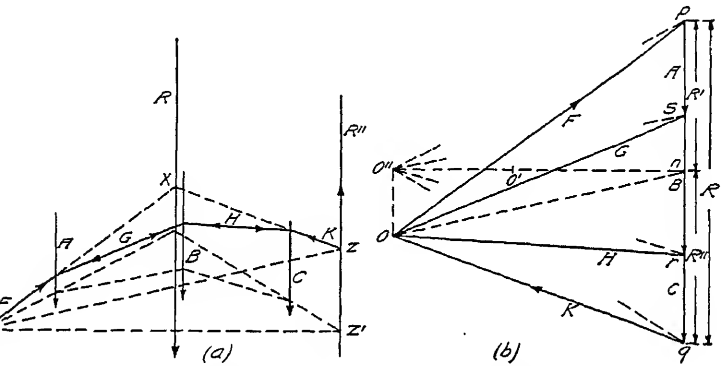


Fig. 226. Equilibrium Polygon with Oblique Closing Line

Nothing was said as to how F, G, H, and K were drawn in 226-a and Fig. 226-b. These forces simply represent one of an ite number of combinations of forces which would produce the e result. The point o is chosen at random, and lines, called are drawn to the extremities of all the forces. The lines of e (A, B, and C) in Fig. 226-b—which is called the *force diagram* e together called the *load line*. The line of forces (F, G, H, K) in Fig. 226-a, together with the closing line $y\,z$, is called an *ibrium polygon.*

Statics of a Linear Arch. We shall assume that the lines in 226 by which we have represented forces F, G, H, and K represent struts which are hinged at their intersections with the forces , and C, which represent loads; and that the two end struts F

and K are hinged at two abutments located at y and z. Then all of the struts will be in compression, and the rays of the force diagram will represent, at the same scale as that employed to represent forces or loads A, B, and C, the compression in each of the struts. In the force diagram, draw a line from o, parallel with the line $y\,z$. It intersects the load line in the point n. Considering the triangle opn as a force diagram, op represents the force F, while pn and on may represent the direction and amount of two forces which will hold F in equilibrium. Therefore pn would represent the amount and direction of the vertical component of the abutment reaction at y, and on would represent the component in the direction of yz. Similarly, we may consider the triangle onq as a force diagram; that nq represents the vertical component R'', and that on represents the component in the direction zy. Since on is common to both of these force triangles, they neutralize each other, and the net resultant of the two forces F and K is the two vertical forces R and R''; but since the resultant R is the resultant of F and K, we may say that R' and R'' are two vertical forces whose combined effect is the equal and opposite of the force R. Although an indefinite number of combinations of forces could begin and end at the points y and z, and could produce equilibrium with the forces A, B, and C, the forces R' and R'' are independent of that particular combination of struts, F, G, H, and K.

Graphical Demonstration of Laws of Statics by Student. The student should test all this work in Statics by drawing figures, very carefully and on a large scale, in accordance with the general instructions as described in the text, and should purposely make some variation in the relative positions and amounts of the forces, from those indicated by the figures. By this means the student will be able to obtain a virtual demonstration of the accuracy of the laws of Statics as formulated. The student should also remember that the laws are theoretically perfect; and when it is stated, for example, that certain lines should be parallel, or that a certain line drawn in a certain way should intersect some certain point, the mathematical laws involved are perfect; and if the drawing does not result in the expected way, it either proves that a blunder has been made, or it may mean that the general method is correct, but that the drawing

Equilibrium Polygon with Horizontal Closing Lines. In Fig. 227, the same forces A, B, and C have been drawn, having the same relative positions as in Fig. 226. The lines of action of the two vertical forces R' and R'' have also been drawn in the same relative position as in Fig. 226. The point n has also been located on the load line in the same position as in Fig. 226. Thus far the lines are a repetition of those already drawn in Fig. 226, the remainder of the figure being omitted, for simplicity. Since the point n in Fig. 226 is the end of the line from the trial pole o, which is parallel to the closing line yz, and since the point n is a definitely fixed point and determines the abutment reactions regardless of the position of the trial pole o, we may draw from n an indefinite horizontal line, such as no', and

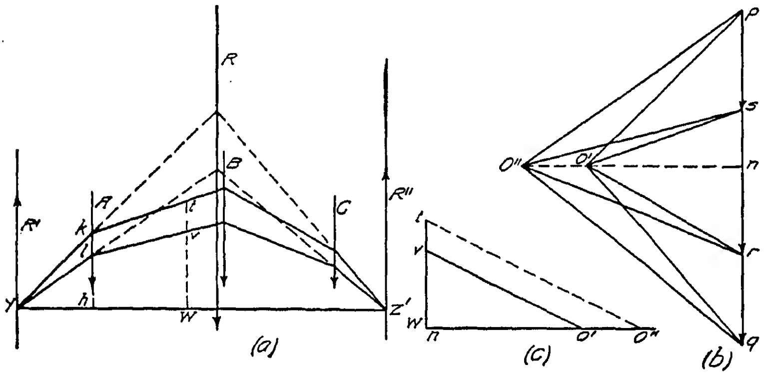


Fig. 227. Equilibrium Polygon with Horizontal Closing Line

we know that the pole of any force diagram must be on this line if the closing line of the corresponding equilibrium polygon is to be a horizontal line. For example, we shall select a point o' on this line, at random. From o' we shall draw rays to the points p, s, r, and q. From the point y, we shall draw a line parallel to $o'p$. Where this line intersects the force A, draw a line parallel to the ray $o's$. Where this intersects the force B, draw a line parallel to the ray $o'r$. Where this intersects the force C, draw a line parallel to the ray $o'q$. This line must intersect the point z', which is on a horizontal line from y. The student should make some such drawing as here described, and should demonstrate for himself the accuracy of this law. This equilibrium polygon is merely one of an infinite number which, if acting as struts, would hold these forces in equilibrium, but it com-

bines the special condition that it shall pass through the points y and z'. There are also an infinite number of equilibrium polygons which will hold these forces in equilibrium and which will pass through the points y and z'.

We may also impose another condition, which is that the first line of the equilibrium polygon shall have some definite direction, such as $y\,l$. In this case the ray from the point p of the force diagram must be parallel to $y\,l$; and where this line intersects the horizontal line $n\,o'$ (produced in this case) is the required position for the pole o''. Draw rays from o'' to s, r, and q, continuing the equilibrium polygon by lines which are respectively parallel to these rays. As a check on the work, the last line of the equilibrium polygon which is parallel to $o''q$ should intersect the point z'. The triangles $y\,k\,h$ and $o'\,p\,n$ have their sides respectively parallel to each other, and the triangles are therefore similar. Their corresponding sides are therefore proportional, and we may write the equation

$$o'n : y\,h :: pn : k\,h$$

Also, from the triangles $y\,l\,h$ and $o''pn$, we may write the proportion

$$o''n : y\,h :: pn : l\,h$$

From these two proportions we may derive the proportion

$$o'n : o''n :: l\,h : k\,h$$

but $o'n$ and $o''n$ are the pole distances of their respective force diagrams, while $k\,h$ and $l\,h$ are intercepts by a vertical line through the corresponding equilibrium polygons. The proportion is therefore a proof, in at least a special case, of the general law that "the perpendicular distances from the poles to the load lines of any two force diagrams are inversely proportional to any two intercepts in the corresponding equilibrium polygons". The above proportions prove the theorem for the intercepts hk and hl. A similar combination of proportions would prove it for any vertical intercept between y and h. The proof of this general theorem for intercepts which pass through other lines of the equilibrium polygon is more complicated and tedious, but it is equally conclusive. Therefore, if we draw any vertical intercept, such as tvw, we may write out the general proportion

In this proportion, if $o''n$ were an unknown quantity, or the position of o'' were unknown, it could be readily obtained by drawing two random lines as shown in Fig. 227-c, and laying off on one of them the distance no', and on the other line the distances vw and tw. By joining v and o' in Fig. 227-c, and drawing a line from t parallel to vo', it will intersect the line no' produced, in the point o''. As a check, this distance to o'' should equal the distance no'' in Fig. 227-b. A practical application of this case, and one that is extensively employed in arch work, is the requirement that the equilibrium polygon shall be drawn so that it shall pass through three points, of which the abutments are two, and some other point (such as v) is the third. After obtaining a trial equilibrium polygon whose closing line passes through the points y and z', the proper position for the pole o'' which shall give the equilibrium polygon that will pass through the point v may be easily determined by the method described above.

The process of obtaining an equilibrium polygon for parallel forces which shall pass through two given abutment points and a third intermediate point may be still further simplified by the application of another property, and without drawing two trial equilibrium polygons before we can draw the required equilibrium polygon. It may be demonstrated that if the pole distance from the pole to the load line is unchanged, all the vertical intercepts of any two equilibrium polygons drawn with these same pole distances are equal. For example, in Fig. 226, a line is drawn vertically upward from o, until it intersects the horizontal line drawn through n in the point o''. This point is the pole of another equilibrium polygon whose closing line will be horizontal, because the pole lies on a horizontal line from the previously determined point n in the load line. Any vertical intercept of this equilibrium polygon will be equal to the corresponding intercept on the first trial equilibrium polygon. Therefore, in order to draw a special equilibrium polygon for a given set of vertical loads, the polygon to pass through two horizontal abutment points and a definite third point between them, we need only draw, first, a trial equilibrium polygon, the rays in the force diagram being drawn through *any* point chosen as a pole. Then if we draw a line from the trial pole parallel with the closing line of this trial equilib-

We next draw a vertical through the point through which the special equilibrium polygon is to pass. The vertical distance of this point above the line joining the abutments is the required intercept of the true equilibrium polygon. The intersection of that vertical with the upper line and the closing line of the trial equilibrium polygon is the intercept of the trial polygon. The pole distance of the true equilibrium polygon is then obtained by the application of Equation (60), by which the pole distances are declared inversely proportional to any two corresponding intercepts of the equilibrium polygons.

Another useful property, which will be utilized later, and which may be readily verified from Figs. 226 and 227, is that, no matter what equilibrium polygon may be drawn, the two extreme lines of the equilibrium polygon, if produced, intersect in the resultant R; therefore, when it is desired to draw an equilibrium polygon which shall pass through any two abutment points, such as $y\,z$ or $y\,z'$, we may draw, from these two abutment points, two lines which shall intersect at any point on the resultant R. We may then draw two lines which will be respectively parallel to these lines from the extremities p and q of the load lines, their intersection giving the pole of the corresponding force diagram.

Equilibrium Polygon for Non-Vertical Forces. The above method is rendered especially simple, owing to the fact that the forces are all vertical. When the forces are not vertical, the method becomes more complicated. The principle will first be illustrated by the problem of drawing an equilibrium polygon which shall pass through the points y, z, and v in Fig. 228. We shall first draw the two non-vertical forces in the force diagram. The resultant R of the forces A and B is obtained as shown in Fig. 224. Utilizing the property referred to above, we may at once draw two lines through y and z which intersect at some assumed point e on the resultant R. Drawing lines from p and q parallel respectively to $e\,z$ and $e\,y$, we determine the point o' as the trial pole for our force diagram. As a check on the drawing, the line joining the intersections b and c should be parallel to the ray $o's$, thus again verifying one of the laws of Statics. If the line $b\,c$ is produced until it inter-

sects the line yz produced, and a line is drawn from the intersection x through the required point v, it will intersect the forces A and B in the points d and g. Then dg will be one of the lines of the required equilibrium polygon. By drawing lines from q and p parallel to yd and zg, we find their intersection o'', which is the pole of the required force diagram. There are two checks on this result: (1) the line so'' is parallel to dg; and (2) the line $o'o''$ is horizontal.

If the line bc is horizontal or nearly so, the intersection (x) of bc and yz produced is at an infinite distance away, or is at least off the drawing. If bc is actually horizontal, the line dg will also be a horizontal line passing through v. When bc is not horizontal, but is

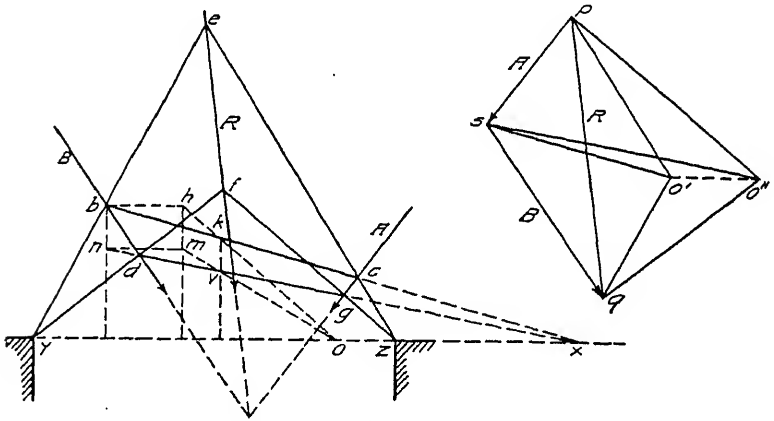


Fig. 228. Equilibrium Polygon through Three Chosen Points

so nearly so that it will not intersect yz at a convenient point, the line dg may be determined as is indicated by the dotted lines in the figure. Select any point on the line yz, such as the point o. Through the given point v, draw a vertical line which intersects the known line bc in the point k. From some point in the line bc (such as the point b), draw the horizontal line bh and the vertical line bn. The line from o through k intersects the horizontal line from b in the point h. From the point h, drop a vertical; this intersects the line ov produced, in the point m. From m, draw a horizontal line which

the force R. Another check on the work, which the student should make, both as a demonstration of the law and as a proof of the accuracy of his work, is to select some other point on the line yz than the point o, and likewise some other point on the line bc than the point b, and make another independent solution of the problem. It will be found that when the drawing is accurate, the new position for the point n will also be on the line dg.

In applying the above principle to the mechanics of an arch, the force A represents the resultant of all the forces acting on the arch on one side of the point v through which the desired equilibrium polygon is required to pass; and the force B is the resultant of all the forces on the other side of that point. A practical illustration of this method will be given later.

VOUSSOIR ARCHES

Definition. A voussoir arch is an arch composed of separate stones, called voussoirs, which are so shaped and designed that the line of pressures between the stones is approximately perpendicular to the joints between the stones. So far as it affects the mechanics of the problem, it is assumed that the mortar in the joints between the voussoirs acts merely as a cushion, and that the mortar has no tensile strength whatever, even if the pressure at any joint should be such as to develop tensile action. It is this feature which constitutes the distinction between a voussoir arch and an elastic arch, which is assumed to be an arch of such material that tensile or transverse stresses may be developed.

Distribution of Pressure between Two Voussoirs. The unit pressure on any joint is assumed to vary in accordance with the location of the center of pressure, as is illustrated in Fig. 229. In the first case, where the center of pressure is over the center of the face of the joint and is perpendicular to it, the pressure will be uniformly distributed, and may be represented, as in Fig. 229-a, by a series of arrows which are all made equal, thus representing equal unit pressures. As the center of pressure varies from the center of the joint, the unit pressure on one side increases and the unit pressure on the other side decreases, as shown in Fig. 229-b. The trapezoid in this diagram has the same area as the rectangle of the first diagram

of the trapezoid. As the center of pressure continues to move away from the center of the joint, the unit pressure on one side becomes greater, and on the other side less, until the center of pressure is at a point $\frac{1}{6}$ of the width of the joint away from the center. In this case (c), the center of pressure is at the extreme edge of the middle third of the joint. The group of pressures illustrated in Fig. 229-c becomes a triangle, which means that the pressure at one side of the joint has become just equal to zero, and that the maximum pressure at the other side of the joint is twice the average pressure. If the line of pressure varies still further from the center of the joint, the diagram of pressures will always be a triangle whose base is always three times the distance of the center of pressure from the nearest edge of the joint. If the *total* pressure on that joint remains constant, then the intensity of pressure on one side of the joint becomes extreme, and may be sufficient to crush the stone. Also, since the elasticity of the stone—or of the mortar between the stones—will cause the stone (or mortar) to yield, the yielding being proportional to the pressure, the joint will *open* at the other side, where there is no pressure. In accordance with this principle of the distribution of pressure, it is always specified that a design for an arch cannot be considered safe unless it is possible to draw a line of pressure—an equilibrium polygon—which shall at every joint pass through the *middle third* of that joint. If the line of pressure at any joint does not pass through the middle third, it means that such a joint will inevitably open, and make a bad appearance, even though the unit pressure on the other end of that joint is not so great that the masonry is actually crushed.

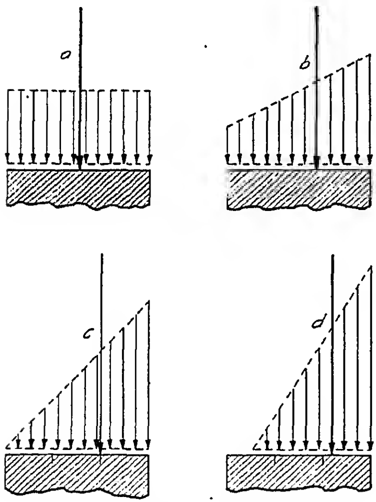


Fig. 229. Diagram Showing Distribution of Pressure

Factor of Safety. Since the actual crushing strength of stone is a rather uncertain and variable quantity, a larger factor of safety is

tion. This factor is usually made ten; and therefore, whenever the line of pressures passes through the edge of the middle third, the average unit pressure on the joint should not be greater than $\frac{1}{20}$ of the crushing strength of the stone.

Quality of Stone. Ultimate values for crushing strength have been given in Table I, Part I, page 6. They vary from about 3,000 pounds per square inch, for a sandstone found in Colorado, up to 28,000 pounds per square inch for a granite found in Minnesota. The weaker stone would hardly be selected for any important work. Usually, a stone whose ultimate strength is 10,000 pounds per square inch or more would be selected for a stone arch. Such a stone could be used with a working pressure of 500 pounds per square inch at any joint, assuming that the line of pressure does not pass outside of the middle third at any joint.

External Forces Acting on an Arch. There is always some uncertainty regarding the actual external forces acting on ordinary arches. The ordinary stone arch consists of a series of voussoirs, which are usually overlaid with a mass of earth or cinders having a depth of perhaps several feet, on top of which may be the pavement of a roadway. The spandrel walls over the ends of the arch, especially when made of squared-stone masonry, also develop an arch action of their own which materially modifies the loading on the arch rings. As this, however, invariably assists the arch, rather than weakens it, no modification of plan is essential on this account. The actual pressure of the earth filling, together with that caused by the live load passing over the arch, on any one stone, is uncertain in very much the same way as the pressure on a retaining wall is uncertain, as previously explained.

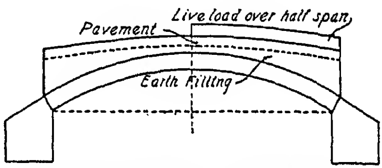


Fig. 230. Diagram Showing Method of Determining Reduced Load Line

The simplest plan is to consider that each voussoir is carrying a load of earth equal to that indicated by lines from the joints in the voussoir vertically upward to the surface. The development of the graphical method makes it more convenient to draw what is called a *reduced load line* on top of the arch, in which the depth of earth

made, Fig. 230. Even the live load on the arch is represented in the same manner, by an additional area on top of the reduced line for the earth pressure, the depth of that area being made in proportion to the intensity of the live load compared with the unit weight of stone. For example, if the earth filling weighs 100 pounds per cubic foot, and the stone of the arch weighs 160 pounds per cubic foot, then each ordinate for the earth load would be $\frac{100}{160}$ of the actual depth of the earth. Likewise, if the live load per square foot on the arch equals 120 pounds, then the area representing the live load would be $\frac{120}{160}$ of a foot, according to the scale adopted for the arch. The weight of the paving, if there is any, should be similarly allowed for. If we draw from the upper end of each joint a vertical line extending to the top of the reduced load line, then the area between these two verticals and between the arch and the load line represents the weight at the scale adopted for the drawing, and at the unit value for the weight per cubic foot—160 pounds per cubic foot, as suggested above—actually pressing on that particular voussoir. A line through the center of gravity of the stone itself gives the line of action of the force of gravity on the voussoir. An approximation to the position of this center of gravity, which is usually amply accurate, is the point which is midway between the two joints, and which is also on the arch curve that lies in the middle of the depth of each voussoir. The center of gravity of the load on the voussoir is approximately in the center of its width. The resultant of two parallel forces, such as V and L, Fig. 231, equals in amount their sum R, and its line of action is between them and at distances from them such that

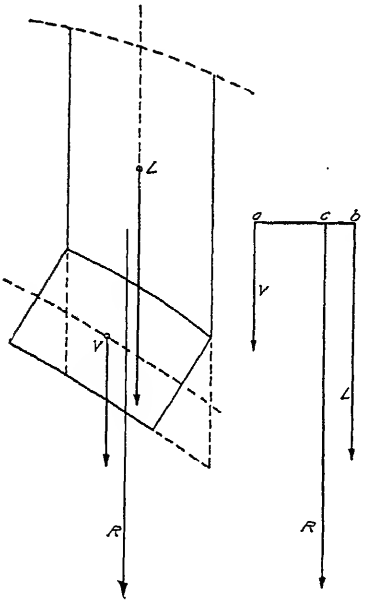


Fig. 231. Graphical Determination of Circular Arch; Span and Rise Being Known

very small that the position of their resultant R can be drawn by estimation as closely as the possible accuracy of drawing will permit, without recourse to the theoretically accurate method just given. The amount of the resultant is determined by measuring the areas, and multiplying the sum of the two areas by the weight per cubic foot of the stone. This gives the weight of a section of the arch ring one foot thick—parallel with the axis of the arch. The area of the voussoir practically equals the length (between the joints of that section) of the middle curve, times the thickness of the arch ring. The area of the load trapezoid equals the horizontal width between the vertical sides, times its middle height. The student should notice that several of the above statements regarding areas, etc., are not theoretically accurate; but, with the usual proportions of the dimensions of the voussoirs to the span of the arch, the errors involved by the approximations are harmless, while the additional labor necessary for a more accurate solution would not be justified by the inappreciable difference in the final results.

Depth of Keystone. The proper depth of keystone for an arch should, theoretically, depend on the total pressure on the keystone of the arch as developed from the force diagram; and the depth should be such that the unit pressure shall not be greater than a safe working load on that stone. But since we cannot compute the stresses in the arch until we know, at least approximately, the dimensions of the arch and its thickness, from which we may compute the dead weight of the arch, it is necessary to make at least a trial determination of the thickness. The mechanics of such an arch may then be computed, and a correction may subsequently be made, if necessary. Usually, the only correction which would be made would be to increase the thickness of the arch, in case it was found that the unit pressure on any voussoir would become dangerously high. Trautwine's Handbook quotes a rule which he declares to be based on a very large number of cases that were actually worked out by himself, the cases including a very large range of spans and of ratios of span to rise. The rule is easily applied, and is sufficiently accurate to obtain a trial depth of the keystone. It will probably be seldom, if ever, that the depth of the keystone, as

determined by this rule, would need to be altered. The rule is as follows:

$$\text{Depth of keystone, in feet} = \frac{\sqrt{\text{rad.} + \text{half-span}}}{4} + 0.2 \text{ ft.} \qquad \textbf{(61)}$$

For architectural reasons, the actual keystone of an arch is usually made considerably deeper than the voussoirs on each side of it, as illustrated in Fig. 223. When computing the maximum permissible pressure at the crown, the actual depth of the voussoirs on each side of the keystone is used as the depth of the keystone; or, perhaps it would be more accurate to say that the extrados is drawn as a regular curve over the keystone, as illustrated in Fig. 233, and then any extra depth which may subsequently be given to the keystone should be considered as mere ornamentation and as not affecting the mechanics of the problem.

ILLUSTRATIVE PROBLEM

Design of Arch with Twenty-Foot Span. The above principles will be applied to the case of an arch having a span of 20 feet and a rise of 3 feet. If this arch is to be a circular arch or a segmental arch, the radius which will fulfill these conditions may be computed as illustrated in Fig. 232. We may draw a horizontal line, at some scale, which will represent the span of 20 feet. At the center of this line we may erect a perpendicular which shall be 3 feet long, at the same scale. Joining the points a and c, and bisecting ac at d, we may draw a line from the bisecting point, which is perpendicular to ac, and this must pass through the center of the required arc. A vertical line through c will also pass through the center of the required arc, and their intersection will give the point o. As a graphical check on the work, a circle drawn about o as a center, and with oc as a radius, should also pass through the points a and b. Since some prefer a numerical solution to determine the radius for a given span and rise, the radius for this case may be computed as follows: The line ac equals the square root of the

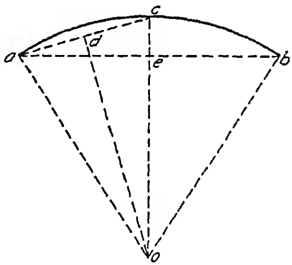
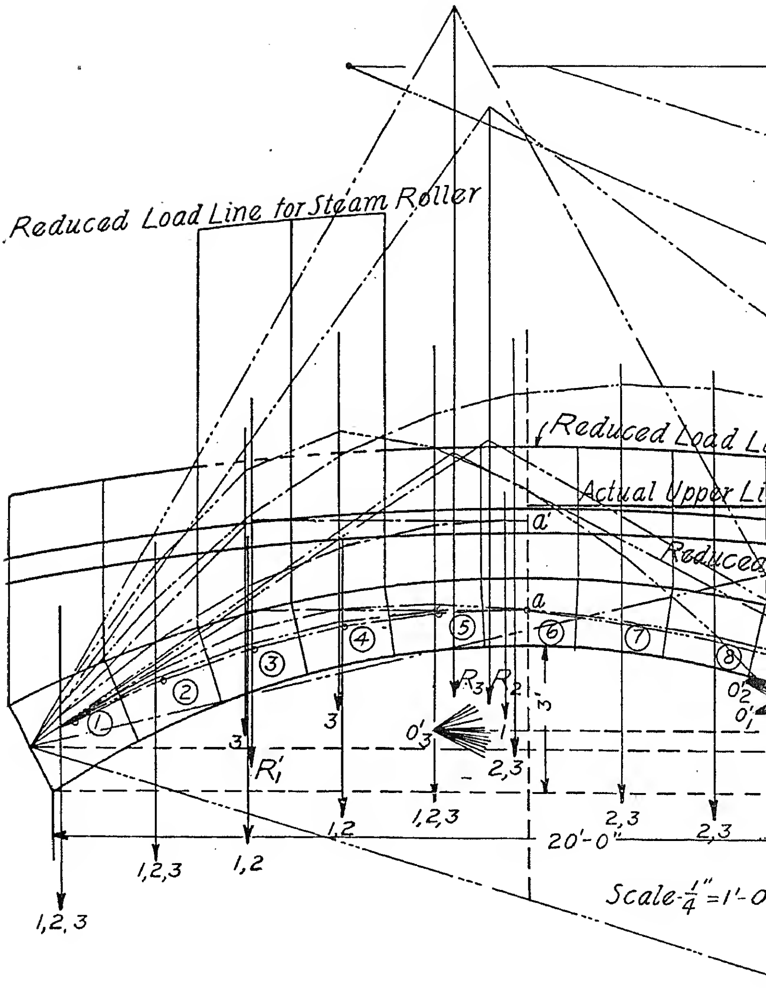


Fig. 232. Diagram of Stresses in Twenty-Foot Arch. Reproduced from Original Drawing at Scale of $\frac{1}{4}$ Inch = 1 Foot



Reduced Load Line for Steam Roller
Reduced Load L
Actual Upper Li
Reduced
a'
a
1
2
3
4
5
6
7
8
R_3
R_2
R'_1
$0'_3$
$0'_2$
$0'_1$
3'
1,2,3
1,2
2,3
20'-0"
Scale-$\frac{1}{4}$"=1'-0

$\sqrt{\overline{ac}^2+\overline{ce}^2}$; but the angle cae equals angle aod, and, from similar triangles, we may write the proportion

$$ao : ad :: ac : ce$$

$$ao=\frac{ad\times ac}{ce}=\frac{1}{2}\frac{\overline{ac}^2}{ce}=\frac{1}{2}\frac{\overline{ae}^2+\overline{ce}^2}{ce}=\frac{1}{2}\frac{\text{half-span}^2+\text{rise}^2}{\text{rise}}$$

This equals numerically, in the above case, $109\div6=18.17$.

Applying the above rule for the depth of the keystone, we would find for this case that the depth should be

$$\begin{aligned}\text{Depth}&=\frac{\sqrt{18.17+10}}{4}+0.2\\&=\frac{5.31}{4}+0.2\\&=1.33+0.2\\&=1.53\text{ ft.}\end{aligned}$$

Since the total pressure on the voussoirs is always greater at the abutment than at the crown, the depth of the stones near the end of the arch should be somewhat greater than the depth of the keystone. We shall therefore adopt, in this case, the dimensions of 18 inches for the depth of the keystone, and 2 feet for the depth at the skewback.

Plotting Reduced Load Line. *Characteristics of Three Loadings.* We shall assume that the earth or cinder fill on top of the arch has a thickness of one foot at the crown, and that it is level on top. We shall also assume that the arch ring is composed of stones which weigh 160 pounds per cubic foot and we shall therefore consider 160 pounds per cubic foot as the unit weight in determining the reduced load line. From the extremities of the extrados, draw verticals until they intersect the upper line of the earth fill. For convenience we shall divide the horizontal distance between these verticals into 11 equal parts, each to be about 2 feet wide, Fig. 233. Draw verticals through these points of division down to the extrados; then draw radial lines from the extrados to the intrados. These lines are drawn radially from a point approximately halfway between the center of the extrados and the center of the intrados. This means that the joints, instead of being exactly perpendicular to

promise between the two. The discrepancy is greatest at the abutments, and approaches zero at the crown. This will divide the arch ring into 11 voussoirs, together with a keystone at the center or crown. Assuming that the earth fill weighs 100 pounds per cubic foot, the lines of division between the 11 sections of the earth fill should each be reduced to $\frac{100}{160}$ or $\frac{5}{8}$ of its actual depth. If we further assume that the pavement is a little over six inches thick, and that its weight is equivalent to six inches of solid stone, we may add a uniform ordinate equal to six inches in thickness (according to the scale adopted), and this gives the total dead load on the arch. We shall assume further a live load amounting to 200 pounds per square foot over the whole bridge. This is equivalent to $\frac{200}{160}$ of a foot, or 1 foot 3 inches of solid masonry over the whole arch. This gives the reduced load line for the condition of loading where the entire arch is loaded with its maximum load.

As another condition of loading, we shall assume that the above load extends only across one-half of the arch. We shall probably find that, owing to the eccentricity of this form of loading, the stability of the arch is in much greater danger than when the entire arch is loaded with a maximum load.

We shall also consider the condition which would be found by running a twenty-ton road roller over the arch. A complete test of all the possible stresses which might be produced under this condition would be long and tedious; but we may make a first trial of it by finding the stresses which would be produced by placing the road roller at one of the quarter-points of the arch—a position which would test the arch almost, if not quite, as severely as any other possible position. Owing to the very considerable thickness of earth fill, as well as the effect of the pavement, the load of the roller is distributed in a very much unknown and very uncertain fashion over a considerable area of the haunch of the arch. The extreme width of such a roller is eight feet; the weight on each of the rear wheels is approximately 12,000 pounds. We shall assume that the weight of each rear wheel is distributed over a width of three feet and a length of four feet, so that the load on the top of the arch under one of the wheels may be considered at the rate of 1,000 pounds per square foot over an area of 12 square feet. For the unit section of the arch one foot

which are four feet in total length. The front roller of the road roller comes between the two rear rollers, and therefore would affect but little, if any, the particular arch ring which we are testing. Not only is it improbable that there would be a full loading of the arch simultaneously with that of a road roller, but it is also true that a full loading would add to the stability of the arch. Yet, in order to make the worst possible condition, we shall assume that the part of the arch which has the road roller is also loaded for the remainder of its length with a maximum load of 200 pounds per square foot; this item alone will take care of the effect of the front roller. A load of 1,000 pounds per square foot is the equivalent of a loading of 6 feet 3 inches of stone; and therefore, if we draw over voussoirs Nos. 3 and 4 a parallelogram having a vertical height above the dead-load line equal to 6 feet 3 inches of stone, and consider a reduced live-load line 15 inches deep ($\frac{200}{160}$ equal to 1.25, or 1 foot 3 inches) over the remainder of that half-span, we have the reduced load line for the third condition of loading.

The loads on each voussoir are scaled from the reduced load line according to the various conditions of loading. The area between the two verticals over each voussoir is measured with all necessary accuracy by multiplying the horizontal width between the verticals by the scaled length of the perpendicular which is midway between the verticals. The weight of the voussoir itself may be computed as accurately as necessary, by multiplying the radial thickness by the length between the joints as measured on the curve lying halfway between the intrados and the extrados.

For example, the load for full loading of the arch which is over voussoir No. 1 is measured as follows: The width between the perpendiculars is 2.0 feet; the height measured on the middle vertical is 4.05 feet; the area is therefore 8.10 feet, which, multiplied by 160, equals 1,296 pounds, which is the load on this voussoir for every foot of width of the arch parallel with the axis. The radial thickness of voussoir No. 1 is 1.90 feet, and the length is 2.15 feet; this gives an area of 4.085 feet, which, multiplied by 160, equals 653.6 pounds. The weight of the voussoir is, therefore, almost exactly one-half that of the live and dead loads above it; therefore, the resultant of these two weights will be almost precisely one-third of the distance between

TABLE XXX

First Condition of Loading

Voussoir No.	Load	Weight of Voussoir	Total
1 and 11	1,296	654	1,950
2 and 10	1,135	592	1,727
3 and 9	1,010	528	1,538
4 and 8	927	483	1,410
5 and 7	880	456	1,336
6	867	45	1,322

Second Condition of Loading

Voussoir No.	Total Load
1	1,950
2	1,727
3	1,538
4	1,410
5	1,336
6	1,122
7	936
8	1,010
9	1,138
10	1,327
11	1,550

Third Condition of Loading

Voussoir No.	Total Load
1	1,950
2	1,727
3	3,138
4	3,010
5	1,336
6	1,122
7	936
8	1,010
9	1,138
10	1,327
11	1,550

loading. By drawing this line, we have the line of action of the resultant of these two forces, and this value is the sum of 1,296 and 654, or 1,950 pounds.

In order to simplify the figure, the arrows representing the lines of force of the loading on the voussoir and the weight of the voussoir have been omitted from the figure, and only their resultant is drawn in. It was of course necessary to draw in these forces in pencil and obtain the position of the resultant, as explained in Fig. 231; and then, for simplicity, only the resultant was inked in.

The loads on the other voussoirs are computed similarly. The numerical values for the loads on the various voussoirs—including the weights of the voussoirs—are given in Table XXX.

For this first condition of loading, the total loads for voussoirs Nos. 7, 8, 9, 10, and 11 will be the same as those for voussoirs 5, 4, 3, 2, and 1, respectively.

The loads for the second condition of loading are found by using the same load on the first five voussoirs, but with only half of the live

dition of loading (1,322 pounds) is reduced by 200 pounds, making it 1,122 pounds. Voussoirs Nos. 7 to 11 are each reduced by 400 pounds.

The loads for the third condition of loading are found by using the same loads as were employed for the second condition, except that for voussoirs Nos. 3 and 4, 1,600 pounds should be added to each load.

Fig. 233 was originally drawn at the scale of ½ inch equal to 1 foot, and with the force diagram at the scale of 1,500 pounds per inch. The photographic reproduction has, of course, changed these scales somewhat. The student should redraw the figure at these scales, and should obtain substantially the same final results.

Drawing the Load Line for the First Condition of Loading. When the load is uniformly distributed over the entire arch, the load is symmetrical, and we need to consider only one-half of the arch. The sections of the load line for the force diagram corresponding to this condition of loading must be drawn as explained in detail on page 386. Since the arch is quite flat, the loading is considered to be entirely vertical. Since the load is symmetrical and the abutments are at the same elevation, we need only draw a horizontal line from the lower end of the half-load line, and select on it a trial position (o_1) for the pole, drawing the rays as previously explained; the trial equilibrium polygon passes through the center vertical at the point a'. Drawing a horizontal line from a' until it intersects the first line (produced) of the trial equilibrium polygon, and drawing through it a vertical line, we have the line of action of the resultant (R_1) of all the forces on that half of the arch. If we draw through a, the center of the keystone, a horizontal line, its intersection with R_1 gives a point in the first line (produced) of the true equilibrium polygon. A line from the upper end of the load line parallel to this first section of the true equilibrium polygon intersects the horizontal line through the middle of the load line at o_1', which is the position of the true pole. Drawing the rays from the true pole to the load line, and drawing the segments of the true equilibrium polygon

The student should carefully check over all these calculations, drawing the arch at the scale of one-half inch to the foot, and the load line of the force diagram at the scale of 1,500 pounds per inch; then the rays of the true equilibrium polygon will represent at that scale the pressure at the joints. Dividing the total depth of any joint by the pressure found at that joint gives the *average* pressure. In the case of the joint at the crown, the total pressure at the joint is 13,900 pounds. The depth of the joint is 1.5 feet, and the area of the joint is 216 square inches; therefore the average unit pressure is 64 pounds per square inch; if it is assumed that the line of pressure passes through either edge of the middle third, then the pressure at the edge of the joint is twice the average, or is 128 pounds per square inch. This is a very low pressure for any good quality of building stone.

Similarly, the maximum pressure at the skewback is scaled from the force diagram as 16,350 pounds; but since the arch is here two feet thick, and the area is 288 square inches, it gives an average pressure of 57 pounds per square inch. Since this equilibrium polygon is supposed to start from the center of this joint, it represents the actual pressure.

Usually, it is only a matter of form to make the test for uniform full loading. Eccentric loading nearly always tests an arch more severely than uniform loading. The ability to carry a full uniform load is no indication of ability to carry a partial eccentric loading, except that if the arch appeared to be only just able to carry the uniform load, it might be predicted that it would probably fail under the eccentric load. On the other hand, if an arch will safely carry a heavy eccentric load, it will certainly carry a load of the same intensity uniformly distributed over it.

Test for the Second Condition, or Loading of Maximum Load over One-Half of the Arch. Since the arch has a dead load over the entire arch, and a live load over only one-half of the arch, the load line for the entire arch must be drawn. The load line for the loaded half of the arch will be identical with that already drawn for the previous case. The load line for the remainder of the arch may be similarly drawn. This case is worked out by precisely the same general method as that already employed in the similar case given in detail

general will give an oblique closing line for the equilibrium polygon. This closing line must be brought down to the horizontal by the method already explained on page 388; then a second trial must be made, in order to shift the polygon so that it shall pass through the middle third at the crown joint. This line should pass through the middle of the crown joint; the real test is then to determine how it passes through the haunches of the arch. As in the previous case, the total pressure at any joint will be determined by the corresponding lines in the force diagram, and the unit pressure at the joint may be determined from the area of the joint and the position of the line of force with respect to the center of the joint. Even though a line of force passed slightly outside of the middle third, it would not necessarily mean that the arch will fail, provided that the maximum intensity of pressure, determined according to the principles enunciated on page 393, does not exceed the safe unit pressure for the kind of stone used.

An inspection of the force diagram with the pole at o_2' shows that the rays are all shorter than those of the force diagram for the first condition of loading—with pole at o_1'. This means that the actual pressure at any joint is less than for the first case; but since the true equilibrium polygon for this case does not pass so near the center of the joints as it does for the first condition of loading, the intensity of pressure at the edges of the joints may be higher than in the first case. However, since the equilibrium polygon for this second case is always well within the middle third at every joint, and since even twice the average joint pressure for the first case is well within the safe allowable pressure on any good building stone, we may know that the second condition of loading will be safe, even without exactly measuring and computing the maximum intensity of pressure produced by this loading.

Test for the Third Condition, Involving Concentrated Load. The method of making this test is exactly similar to that previously given; but, on account of a load eccentrically placed, the force diagram will be more distorted than in either of the cases previously given, and there is greater danger that the arch will prove to be unstable on such a test. An inspection of the equilibrium polygon for this case shows that the critical point is the joint between voussoirs Nos. 3 and 4. This is what might be expected, since it is the joint under

the heavy concentrated load. The ray in the force diagram which is parallel to the section of the equilibrium polygon passing through this joint is the ray which reaches the load line between loads 3 and 4. This ray, measured at the scale of 1,500 pounds per square inch, indicates a pressure of 15,625 pounds on the joint. The line of pressure is $4\frac{3}{4}$ inches from the upper edge of the joint; it is outside of the middle third; and therefore the joint will probably open somewhere under this loading. According to the theory of the distribution of pressure over a stone joint, the pressure will be maximum on the upper edge of this joint, and will be zero at three times $4\frac{3}{4}$ inches, or 14.25 inches, from the upper edge. The area of pressure for a joint 12 inches wide will be 14.25×12, or 171 square inches. Dividing 171 into 15,625, we have an average pressure of 91 pounds, or a maximum pressure of twice this, or 182 pounds, per square inch at the edge of the joint. But this is so safe a working pressure for such a class of masonry as cut-stone voussoirs, that the arch certainly would not fail, even though the elasticity of the stone caused the joint to open slightly at the intrados during the passage of the steam roller.

Correcting a Design. The above general method of testing an arch consists of first designing the arch, and then testing it to see whether it will satisfy all the required conditions. In case some condition of loading is found which will cause the line of pressure to pass outside of the middle third or to introduce an excessive unit pressure in the stones, it is theoretically necessary to begin anew with another design, and to make all the tests again on the basis of a new design; but it is usually possible to determine with sufficient closeness just what alterations should be made in the design so that the modified design will certainly satisfy the required conditions. For example, if the line of pressure passes on the upper side of the middle third at the haunches of the arch, a thickening of the arch at that point, until the line of pressure is within the middle third of the revised thickness, will usually solve the difficulty. The effect of the added weight on the haunch of the arch will be to make the line of pressure move upward slightly; but the added thickness can allow for this. As another illustration, the unit pressure, as determined for the crown of the arch, might be considerably

necessity to thicken the arch, not only at the center, but also throughout its entire length.

For example, in the above numerical case, although it is probably not really necessary to alter the design, the arch might be thickened on the haunches, say, 3 inches. This would add to the weight on the haunches one-fourth of the difference of the weights per cubic foot of stone and earth, or $\frac{1}{4}$ (160 − 100), or 15 pounds per square foot. This is so utterly insignificant compared with the actual total load of about 750 pounds per square foot, that its effect on the line of pressure is practically inappreciable, although it should be remembered that the effect, slight as it is, will be to raise the line of pressure. A thickening of 3 inches will leave the line of pressure nearly $7\frac{3}{4}$ inches—or, say, $7\frac{1}{2}$ inches, to allow generously for the slight raising of the line of pressure—from the extrados, while the thickness of the arch is increased from 19 inches to 22 inches. But the line of pressure would now be within the middle third.

Location of True Equilibrium Polygon. In the above demonstration, it is assumed that the true equilibrium polygon will pass through the center of each abutment, and also through the center of the keystone; and the test then consists in determining whether the equilibrium polygon which is drawn through these three points will pass within the middle third at every joint, or at least whether it will pass through the joints in such a way that the maximum intensity of pressure at either edge of the joint shall not be greater than a safe working pressure. With any system of forces acting on an arch, it is possible to draw an infinite number of equilibrium polygons; and then the question arises, which polygon, among the infinite number that can be drawn, represents the true equilibrium polygon and will represent the actual line of pressure passing through the joints. On the general principle that forces always act along the line of least resistance, the pressure acting through any voussoir would tend to pass as nearly as possible through the center of the voussoir; but since the forces of an equilibrium polygon, which represent a combination of lines of pressure, must all act simultaneously, it is evident that the line of pressure will pass through the voussoirs by a course which will make the summation of the intensity of pressures at the various joints a minimum. It is not only possible, but

center of the keystone, but at some point a little above or below, through which a polygon may be drawn which will give a less summation of pressures than those for a polygon which does pass through the point a. The value and safety of the method given above lie in the fact that the true equilibrium polygon always passes through the voussoirs in such a way that the summation of the intensities of the pressures is the least possible combination of pressures; and, therefore, any polygon which can be drawn through the voussoirs in such a way that the pressures at all the joints are safe merely indicates that the arch will be safe, since the true combination of pressures is something less than that determined. In other words, the true system of pressures is never greater, and is probably less, than the system as determined by the equilibrium polygon, which is assumed to be the true polygon.

When an equilibrium polygon for eccentric loading passes through the arch at some distance from the center of the joint at one part of the arch, and very near the center of the joint in all other sections, it can be safely counted on, that the true polygon passes a little nearer the center at the most unfavorable portion, and a little farther away from the center at some other joints where there is a larger margin of safety. For example, the true equilibrium polygon for the third condition of loading, Fig. 233, probably passes a little nearer the center on the left-hand haunch, and a little farther away from the center on the right-hand haunch, where there is a larger margin; in other words, the whole equilibrium polygon is slightly lowered throughout the arch. No definite reliance should be placed on this allowance of safety; but it is advantageous to know that the margin exists, even though that margin is very small. The margin, of course, would reduce to zero in case the equilibrium polygon chosen actually represented the true equilibrium polygon. While it would be convenient and very satisfactory to be able to obtain always the true equilibrium polygon, it is sufficient for the purpose to obtain a polygon which indicates a safe condition when we know that the true polygon is still safer.

Design of Abutments. *Pressure Diagram.* The force diagram of Fig. 233, which shows the pressures between the voussoirs of the

the maximum pressure against the abutment comes against the left-hand abutment under the third condition of loading, when the concentrated load is on the left-hand side of the arch. Although the first condition of loading does not create so great a pressure against the left-hand abutment, yet the angle of the line of pressure is somewhat flatter, and this causes the resultant pressure on the base of the abutment to be slightly nearer the rear toe of the abutment. It is therefore necessary to consider this case, as well as that of the third condition of loading.

Failure of Abutments. An abutment may fail in three ways: (1) by sliding on its foundations; (2) by tipping over; and (3) by crushing the masonry. The possibility of failure by crushing the masonry at the skewback may be promptly dismissed, provided the quality of the masonry is reasonably good, since the abutment is always made somewhat larger than the arch ring, and the unit pressure is therefore less. The possibility of failure by the crushing of the masonry at the base, owing to an intensity of pressure near the rear toe of the abutment, will be discussed below. The possibility that the abutment may slide on its foundations is usually so remote that it hardly need be considered. The resultant pressure of the abutment on its subsoil is usually nearer to the perpendicular than is the angle of friction; and in such a case, there will be no danger of sliding, even if there is no backing of earth behind the abutment, such as is almost invariably found.

The test for possible tipping over or crushing of the masonry, due to an intensity of pressure near the rear toe, must be investigated by determining the resultant pressure on the subsoil of the abutment. This is done graphically by the method illustrated in Fig. 234. This is an extension of the arch problem already considered. The line bc gives the angle of the skewback at the abutment, while the lines of force for the pressures induced by the first and third conditions of loading have been drawn at their proper angle. In common with the general method used in designing an arch, it is necessary to design first an abutment which is assumed to fulfill the conditions, and then to test the design to see whether it is actually suitable. The cross section $abcde$ has been assumed as the cross section of solid masonry for the abutment. The problem, therefore,

senting the weight of the abutment. It will be proved that this force passes through the point o_5, and it therefore intersects the pressure on the abutment for the first condition of loading, at the point k. The weight of a section of the abutment one foot thick parallel with the axis of the arch— is computed, as detailed below, to weigh 19,500 pounds, while the pressure of the arch is scaled from Fig. 233 as 16,350 pounds. Laying off these forces on these two lines at the scale of 5,000 pounds per inch, we have the resultant, which

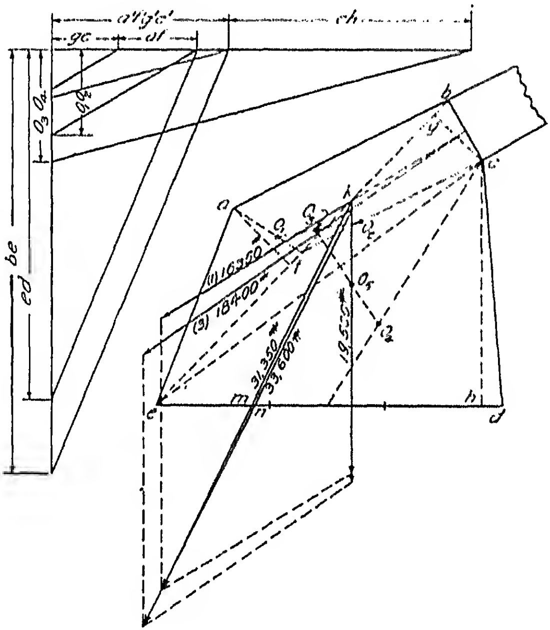


Fig. 234. Diagram of Forces Acting on Abutments

intersects the base at the point m, and which scales 31,350 pounds. Similarly, the resultant of the weight of the abutment and the line of pressure for the third condition of loading intersects the base at the point n, and scales 33,600 pounds. These pressures on the base will be discussed later.

Line of Action. The line of action and the amount of the weight of a unit section of the abutment are determined as follows: The center of gravity of the pentagon *abcde* is determined by

triangles abe and bce. By bisecting the base be and drawing lines to the vertices a and c, and trisecting these lines to the vertices, we determine the points o_1 and o_2, which are the centers of gravity, respectively, of the two triangles. The center of gravity of the combination of the two triangles must lie on the line joining o_1 and o_2, and must be located on the line at distances from each end which are inversely proportional to the areas of the triangles. Since the triangles have a common base be, their areas are proportional to their altitudes af and gc. In the diagram at the side, we may lay off in succession, on the horizontal line, the distances gc and af. On the vertical line, we lay off a distance equal to o_1o_2. By joining the lower end of this line with the right-hand end of the line af, and then drawing a parallel line from the point between gc and af, we have divided the distance o_1o_2 into two parts which are proportional to the two altitudes af and gc. Laying off the shorter of these distances toward the triangle abe (since its greater altitude shows that it has the greater area), we have the position of o_3, which is the center of gravity of the two triangles combined. The area $abce$ is measured by one-half the product of eb and the sum of af and gc. The triangle cde is measured by one-half the product of the base ed by the altitude ch. If we lay off be as a vertical line in the side diagram, and also the line ed as a vertical line, and join the lower end of ed with the line which represents the sum of gc and af, and then draw a line from the lower end of be, parallel with this other line, we have two similar triangles from which we may write the proportion

$$ed : (gc+af) :: be : a'f'g'c'$$

Since the product of the means equals the product of the extremes, we find that $(gc+af)\times be = ed\times a'f'g'c'$; but $\frac{1}{2}\,(gc+af)\times be$ equals the combined area of the two triangles, and therefore the line $a'f'g'c'$ is the height of an equivalent triangle whose base equals ed; therefore the area of these two combined triangles is to the area of the triangle cde as the equivalent altitude $a'f'g'c'$ is to the altitude ch of the triangle cde. By bisecting the base ed, and drawing a line

areas of the two combined triangles and the triangle *cde*. These areas are proportional to the altitudes as determined above; therefore, by laying off in the side diagram the line o_3o_4, and drawing a line from its lower extremity to the right-hand extremity of the line *ch*, and then drawing a parallel line from the point between *a'f'g'c'* and *ch*, we divide the line o_3o_4 into two parts which are proportional to these altitudes. The line *ch* is the greater altitude, and the triangle *cde* has the greater area; therefore, the point o_5 is nearer to the point o_4 than it is to the point o_3, and the shorter of these two sections is laid off from the point o_4. This gives the point o_5, which is the center of gravity of the entire area of the abutment.

Weight of Unit Section. The actually computed weight of a unit section of the abutment is determined by multiplying the sum of *a'f'g'c'* and *ch* by the base *cd*. Since this masonry is assumed to weigh 160 pounds per cubic foot, the product of these scaled distances, measured at the scale of ½ inch equal to one foot, which was the scale adopted for the original drawing, shows that the section one foot thick has a weight of 19,500 pounds. Laying off this weight from the point *k*, and laying off the pressure for the first condition of loading, 16,350 pounds, at the scale of 5,000 pounds per inch, and forming a parallelogram on these two lines, we have the resultant of 31,350 pounds as the pressure on the base of the abutment, that pressure passing through the point *m*.

Line of Pressure. The intersection of the weight of the abutment with the line of pressure for the third condition of loading is a little below the point *k*; and we similarly form a parallelogram which shows a resulting pressure of 33,600 pounds, passing through the base at the point *n*. It is usually required that such a line of pressure shall pass through the middle third of the abutment; but there are other conditions which may justify the design, even when the line of pressure passes a little outside of the middle third.

The point *n* is 2.85 feet from the point *c*. According to the theory of pressures enunciated on page 393, it may be considered that the pressure is maximum at the point *c*, and that it extends backward toward the point *d* for a distance of three times *cn*, or a distance of 8.55 feet. This would give an average pressure of 3,930 pounds per

might or might not be greater than the subsoil could endure without yielding. Since this pressure is equivalent to about 55 pounds per square inch, there should be no danger that the masonry itself would fail; and, if the subsoil is rock or even a hard, firm clay, there will be no danger in trusting such a pressure on it.

Effect of Back Pressure. Another very large item of safety which has been utterly ignored, but which would unquestionably be present, is the pressure of the earth back of the abutment. The effect of the back pressure of the earth would be to make the line which represents the resultant pressure on the subsoil more nearly vertical, and to make it pass much more nearly through the center of the base *ed*. This would very much reduce the intensity of pressure near the point *e*, and would reduce very materially the unit pressure on the subsoil. Cases, of course, are conceivable, in which there might be no back pressure of earth against the rear of the abutment. In such cases, the ability of the subsoil to withstand the unit pressure at the rear toe of the abutment —near the point *e*— must be more carefully considered. In order that the investigation shall be complete, it should be numerically determined whether the lower pressure, 31,350 pounds, passing through the point *m*, might produce a greater intensity of pressure at the point *e* than the larger pressure passing through the point *n*.

Various Forms of Abutments. The abutment described above is the general form which is adopted very frequently. The front face *cd* is made with a batter of one in twelve. The line *ba* slopes backward from the arch on an angle which is practically the continuation of the extrados of the arch. The total thickness of the abutment *de* must be such that the line of pressure will come nearly, if not quite, within the middle third. The line *ea* generally has a considerable slope, as is illustrated. When the subsoil is very soft, so that the area of the base is necessarily very great, the abutment is sometimes made hollow, with the idea of having an abutment with a very large area of base, but one which does not require the full weight of so much masonry to hold it down; and therefore economy is sought in the reduction of the amount of masonry. As such a hollow abutment would require a better class of masonry than could be used for a solid block of masonry, it is seldom that there is any economy in

withstand a very great lateral thrust from the arch, there is never any danger that the resultant pressure of the abutment on the subsoil will approach the front toe of the arch, as is the case in the abutment of a steel bridge, which has little or no lateral pressure from the bridge, but which is usually subjected to the pressure of the earth behind it. These questions have already been taken up under the subject of abutments for truss bridges, in Part II.

VOUSSOIR ARCHES SUBJECTED TO OBLIQUE FORCES

Determination of Load on a Voussoir. The previous determinations have been confined to arches which are assumed to be acted on solely by vertical forces. For flat segmental arches, or even for elliptical arches where the arch is very much thickened at each end so that the virtual abutment of the arch is at a considerable distance above the nominal springing line, such a method is sufficiently accurate, and it has the advantage of simplicity of computation; but where the arch has a very considerable rise in comparison with its span, the pressure on the extrados, which is presumably perpendicular to the surface of the extrados, has such a large horizontal component that the horizontal forces cannot be ignored. The method of determining the amount and direction of the force acting on each voussoir is illustrated in Fig. 235. The reduced load line, found as previously described, is indicated in the figure. A trapezoid represents the loading resting on the voussoir ac. The line df represents, at some scale, the amount of this vertical loading. Drawing

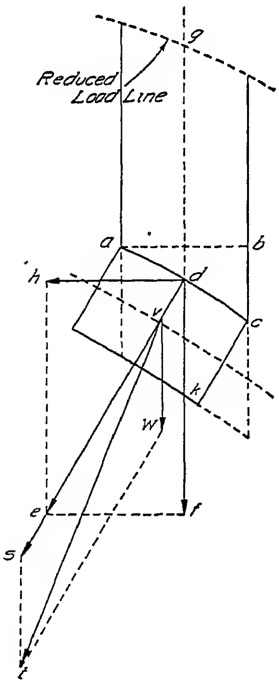


Fig. 235. Diagram of Resultant of Oblique Pressures

effect of the friction of the earth on the voussoir, which will invariably reduce the horizontal component by some uncertain amount. The actual horizontal component is an indeterminate quantity except on the basis of assumptions which are perhaps unwarranted.

Drawing a vertical line through the center of gravity of the voussoir, and producing it, if necessary, until it intersects ed in the point v, we may lay off vw to represent, at the same scale, the weight of the voussoir. Making vs equal to de, we find vt as the resultant of the forces; and it therefore measures, according to the scale chosen, the amount and direction of the resultant of the forces acting on that voussoir. Although the figure apparently shows the line de as though it passed through the center of gravity of the voussoir, and although it generally will do so very nearly, it should be remembered that de does not necessarily pass through the center of gravity of the voussoir.

A practical graphical method of laying off the line vt to represent the actual resultant force is as follows: The reduced load line, drawn as previously described, gives the line for a loading of solid stone, which would be the equivalent of the actual load line. If this loading has a unit value of, say, 160 pounds per cubic foot, and if the horizontal distance ab is made 2 feet for the load over each voussoir, then each foot of height (at the same scale at which ab represents 2 feet) of the line gd represents 320 pounds of loading. If the voussoir were actually a rectangle, then its area would be equal to that of the dotted parallelogram vertically under ac, and its area would equal $ab \times dk$; and in such a case dk would represent the weight of that voussoir, and the force vw could be scaled directly equal to dk, without further computation. The accuracy of this method, of course, depends on the equality of the dotted triangle below c and that below a. For voussoirs which are near the crown of the arch, the error involved by this method is probably within the general accuracy of other determinations of weight; but near the abutment of a full-centered arch, the inaccuracy would be too great to be tolerated, and the area of the voussoir should be actually computed. Dividing the area by 2 (or the width ab), we have the equivalent height in the same terms at which gd represents the

Application to Full=Centered Arch. *Assumed Dimensions.* We shall assume for this case a full-centered circular arch whose intrados has a radius of 15 feet. The depth of the keystone computed according to the rule given in Equation (61), would be 1.57 feet, which is practically 19 inches. By drawing first the intrados of the arch as a full semicircle, as in Fig. 236, and then laying off the crown thickness of 19 inches, we find by trial that a radius of 20 feet for the extrados will make the arch increase to a thickness of about $2\frac{1}{2}$ feet at a point 45 degrees from the center, which is usually a critical point in such arches. We shall therefore draw the extrados with a radius of 20 feet, the center point being determined by measuring 20 feet down from the top of the keystone. We shall likewise assume that this arch is one of a series resting on piers which are 4 feet thick at the springing line.

By drawing a portion of the adjoining arch, we find that its extrados intersects the extrados of the arch considered, at a point about 7 feet 6 inches above the pier. By drawing a line from this point toward the *center for joints,* lying about midway between the center for the extrados and the center for the intrados, we have the line for the joint which is virtually the skewback joint and the abutment of the arch.

Assumed Earth and Track Loads. The center of the pier is precisely 17 feet from the center of the arch. We shall assume that the arch is overlaid with a filling of earth or cinders which is 18 inches thick at the crown, and that it is level. Drawing a horizontal line to represent the top of this earth filling, we may divide this line into sections which are 2 feet wide, commencing at the vertical line through the center of the pier. Extending this similarly to the other side of the arch, we have eight sections of loading on each side of the keystone section. Drawing lines from the points where these verticals between the sections intersect the extrados, toward the center for joints, previously determined, we have the various joints of the voussoirs. Assuming, as in the previous numerical problem, that the cinder fill weighs 100 pounds per cubic foot, and that the stone weighs 160 pounds per cubic foot, we determine the reduced load line for the top of the earth fill over the entire arch.

We shall assume that the arch carries a railroad track and a

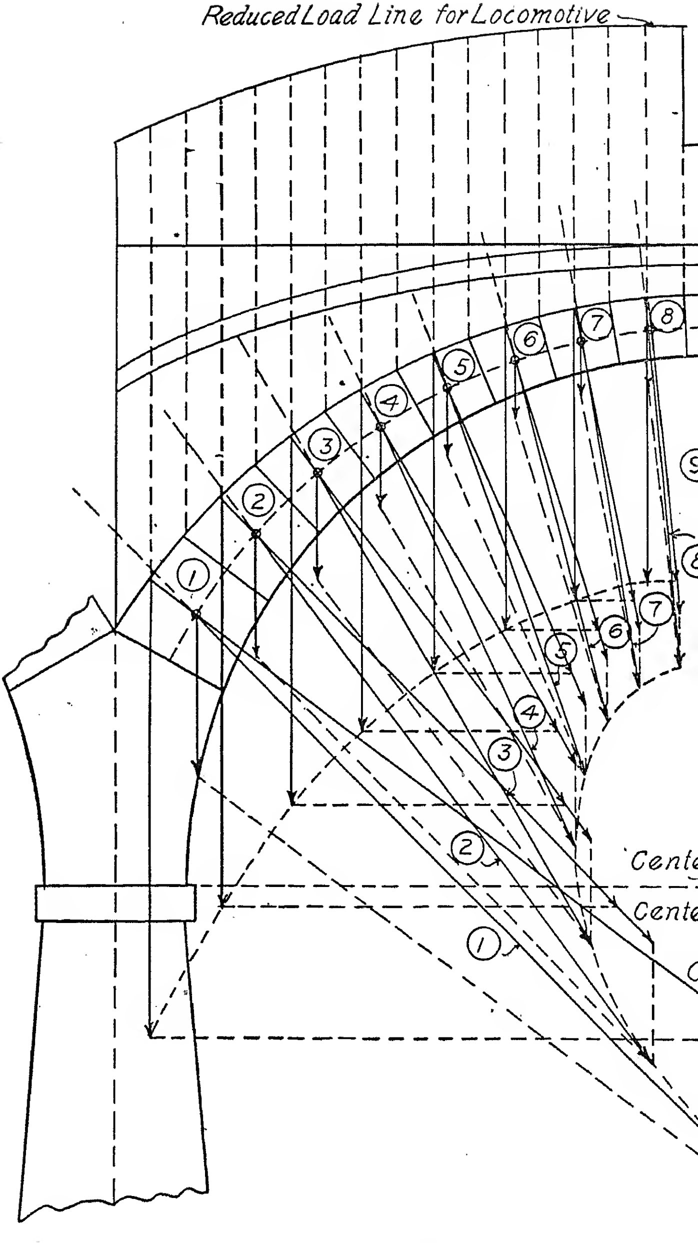


Fig. 236. Diagram of Resultant Forces Acti

heavy class of traffic. The weight of roadbed and track may be computed as follows: The ties are to be 8 feet long; the weight of the roadbed and track (and also the live load) is assumed to be distributed over an area 8 feet wide.

Two rails at 100 pounds per yd. will weigh, per sq. ft. of surface.......	8.4 lb.
Oak ties, weighing 150 pounds per tie, will weigh, per sq. ft. of surface..	9.4 lb.
Weight of ballast, at 100 pounds per cu. ft.; average depth 9 in.......	75.0 lb.
Total weight	92.8 lb.

This is the equivalent of 0.58 foot depth of stone, and we therefore add this uniform depth to the reduced load line for the earth.

Assumed Live Load. A 50-ton freight-car, fully loaded, will weigh 134,000 pounds; with a length between bumpers of 37 feet, this will exert a pressure of about 450 pounds per square foot on a strip 8 feet wide. This is equivalent to 2.8 feet of masonry. We shall therefore consider this as a requirement for uniform loading over the whole arch.

Summary of Conditions of Loading. It would be more precise to consider the actual wheel loads for the end trucks of two such cars which are immediately following each other; but since the effect of this would be even less than that of the calculation for a locomotive, which will be given later, and since the deep cushion of earth filling will largely obliterate the effect of concentrated loads, the method of considering the loading as uniformly distributed will be used. We therefore add the uniform ordinate equal to 2.8 feet over the whole arch. We shall call this the *first condition of loading.*

We shall assume for the concentrated loading, a consolidation locomotive with 40,000 pounds on each of the four driving axles, spaced 5 feet apart. This means a wheel base 15 feet long; and we shall assume that this extends over voussoirs 1 to 8 inclusive, while the loading of 450 pounds per square foot is on the other portion of the arch. A weight of 40,000 pounds on an axle, which is supposed to be distributed over an area 5 feet long and 8 feet wide, gives a pressure of 1,000 pounds per square foot, or it would add an ordinate of 6.33 feet of stone; these ordinates are added above the load line representing the load of the roadbed and track. We shall call this the *second condition of loading.*

Method of Computing Loads. The load for each voussoir is

pressure on the voussoir is determined by drawing a line toward the extrados center from the intersection of the vertical through the trapezoid of loading with the extrados. The length of that vertical is laid off below that point of intersection; then a horizontal line drawn from the lower end of the vertical intersects the line of force at a point which measures the amount of that pressure on the voussoir. The area of the voussoir is determined as described on page 418; and the resultant of the loading and the weight of the voussoir is obtained. This is indicated as force No. 1 in Fig. 236. In this case, it includes the locomotive loading on the left-hand side of the arch. The forces acting on voussoirs Nos. 2, 3, 4, 5, 6, 7, and 8 are similarly determined. The forces on voussoirs Nos. 9 to 17, inclusive, on the basis of the uniformly distributed load equal to 450 pounds per square foot, are also similarly determined. The loads on voussoirs Nos. 10 to 17, inclusive, will be considered to measure the loads on voussoirs Nos. 8 to 1, inclusive, for the first condition of loading. The loading with the locomotive over voussoirs Nos. 1 to 8, and cars over voussoirs Nos. 9 to 17, constitutes the second condition of loading.

As described above, the arrows representing the forces in Fig. 236 are drawn at a scale such that each $\frac{3}{8}$ of an inch represents 2 cubic feet of masonry, or 320 pounds; therefore every inch will represent the quotient of 320 divided by $\frac{3}{8}$, or 853 pounds per lineal inch. The practical method of making a scale for this use is illustrated in the diagram in the upper right-hand corner of Fig. 236. We may draw a horizontal line as a scale line, and lay off on it, with a decimal scale, a length *ca* which represents, at some convenient scale, a length of 800. Drawing the line *ab* at any convenient angle, we lay off from the point *c* the length *cb* to represent 853 at the same scale as that used for *ca*. The line *cd* is then laid off to represent 7,000 units at the scale of 800 units per inch. By drawing a line from *d* parallel to *ba*, we have the distance *ce*, which represents 7,000 units at the scale of 853 units per inch. By trial, a pair of dividers may be so spaced that it steps off precisely seven equal parts for the distance *ce*; or the line *ce* may also be divided into equal parts by laying off on *cd* to the decimal scale, the seven equal

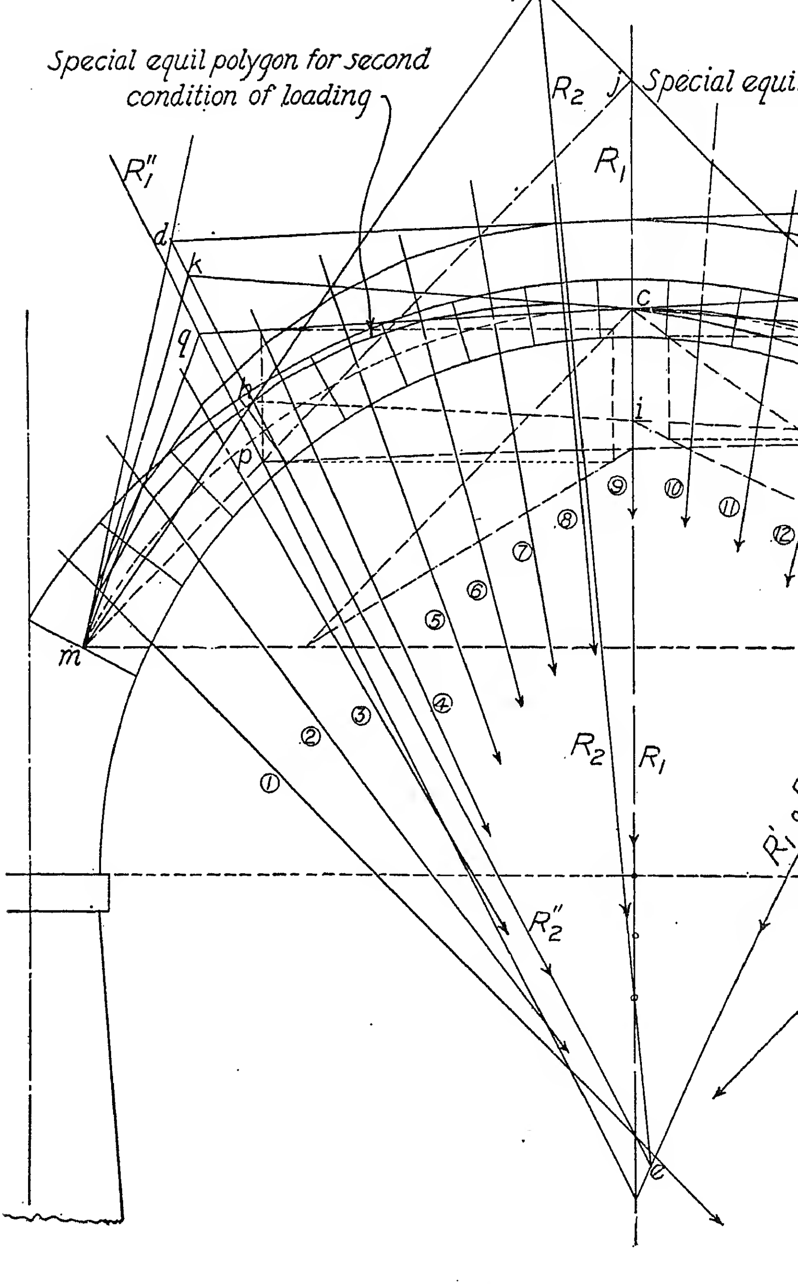


Fig. 237. Diagram of Pressures on V

The last division may be similarly divided into 10 equal parts, which will represent 100 pounds each. Using dividers, the resultant force on each voussoir from No. 1 to No. 17 may be scaled off as follows:

1	7,825	7	3,170	13	2,400
2	5,970	8	3,040	14	2,905
3	4,940	9	1,880	15	3,570
4	4,190	10	1,910	16	4,420
5	3,725	11	2,040	17	6,005
6	3,380	12	2,200		

Graphical Check. Note the three dotted curves in the lower part of Fig. 236, which have been drawn through the extremities of the forces. The object in drawing these three curves is merely to note the uniformity with which the ends of these arrows form a regular curve. If it were found that one of the forces did not pass through this curve, it would probably imply a blunder in the method of determining that particular force. Even if such curves are not actually drawn in, it is well to observe that the points do come on a regular curve, as this constitutes one of the checks on the graphical solution of problems.

Fig. 236 is merely the beginning of the problem of determining the stresses in the arch. In order to save the complication of the figure, the arch itself and the resultant forces (1 to 17) are repeated in Fig. 237, the direction, intensity, and point of application of these forces being copied from one figure to the other.

Pressure Diagram for Both Conditions of Loading. Forces Nos. 1 to 17 are drawn in the force diagram of Fig. 237 at the scale of 4,000 pounds per inch. Forces 1 to 8, inclusive, have a resultant whose direction is given by the line marked R_1'' which joins the extremities of forces 1 to 8. Similarly, the direction of the resultant (R_1' or R_2') of forces 9 to 17, inclusive, is given by the line which joins the extremities of this group. The direction of the resultant of all the forces, Nos. 1 to 17, is given by the line joining the extremities of these forces in the force diagram, this resultant being marked R_2. By choosing a pole at random (the point o_2' in the force diagram), drawing rays to the forces, and beginning at the left-hand abutment, we may draw the trial equilibrium polygon, which passes

polygon which is between forces 8 and 9—and which is parallel to the ray which reaches the load line between forces 8 and 9—it intersects the first and last lines of the trial equilibrium polygon at the points b and d. The point b is, therefore, a point on the resultant R_2' of forces Nos. 9 to 17, inclusive; and by drawing a line parallel to the force R_2' in the force diagram, we have the actual line of action of the resultant.

Similarly, the line of action of the force R_2'' is determined by drawing from the point d a line parallel to R_2'' in the force diagram. Their intersection at the point e gives a point in the line of action of the resultant of the whole system of forces, R_2; and by drawing from the point e a line parallel to R_2 of the force diagram, we have the line of action of R_2. We select a point f at random on the resultant R_2, and join the point f with the center of each abutment. By drawing lines from the extremities of the load line parallel to these two lines from f, they intersect at the point o_2''. A horizontal line through o_2'' is therefore the locus of the pole of the true equilibrium polygon passing through the center of both abutments. The line fn intersects R_2' in the point g, and the line fm intersects the force R_2'' in the point h. The intersection of gh with the vertical through the center—the point i—is the trial point which must be raised up to the point c, which is done by the method illustrated on page 390. The application of this method gives the line kl, passing through c; and the line ln is therefore the first line of the special equilibrium polygon for the complete system of forces from No. 1 to No. 17; and the line km is similarly the last line of that polygon. By drawing lines from the extremities of the load line, parallel to ln and km, we find that they intersect at the point o_2''', which is the pole of the special equilibrium polygon passing through n, c, and m, for the complete system of forces Nos. 1 to 17.

As a check on the work, the intersection of these lines from the ends of the load line, parallel to ln and km, must be on the horizontal line passing through o_2''. By drawing rays from the new pole o_2''' to the load line, and completing the special equilibrium polygon, we should find, as a double check on the work, that both of these partial polygons starting from m and n should pass through the point c; and also that the section of the polygon between forces Nos. 8

and 9 lies on the line kl. This gives the special equilibrium polygon for the system of forces Nos. 1 to 17, which corresponds with the second condition of loading, as specified above.

The first condition of loading is given by duplicating about the center, in the force diagram, the system of forces from No. 17 to No. 9, inclusive. Since this system of forces is symmetrical about the center, we know that its resultant R_1 passes through the center of the arch, and that it must be a vertical force. We may draw from the middle of force No. 9 a horizontal line, and also draw a vertical from the lower end of the load line. Their intersection is evidently at the center of the resultant R_1, which is, therefore, carried above this horizontal line for an equal amount. Joining the upper end of R_1 with the upper end of force No. 9, we have the direction and amount of the force R_1''. The intersection of ng with the force R_1 at the point j, gives a point which, when joined with the point m, gives one line of a trial equilibrium polygon passing through the required points m and n, but which does not pass through the required point c. The intersection of jm with the force R_1'' at the point p, gives us the line pg, which is the same kind of line for this trial polygon as the line hg was for the other.

By a similar method to that used before and as described in detail on page 391, we obtain the line qr passing through c, which gives us also the section of our true equilibrium polygon between forces Nos. 8 and 9. The line rn also gives us that portion of the true equilibrium polygon for this system of loading, from the point n up to the force No. 17.

By drawing a line from the lower end of the load line, parallel to nr, until it intersects the horizontal line through the middle of force No. 9 at the point o_1', we have the pole of the special equilibrium polygon for this system of loading, which is the first condition of loading. The rays are drawn from o_1' only to the forces from No. 9 to No. 17, inclusive, and the special equilibrium polygon is completed between n and c by drawing them parallel to these rays.

On account of the symmetry of loading, we know that the equilibrium polygon would be exactly similar on the left-hand side of the arch. In discussing these equilibrium polygons, we must therefore remember that of the two equilibrium polygons lying between the

line represents the line of pressure for a uniform loading over the whole arch—the first condition of loading—while the lower line on the right-hand side, and also the one equilibrium polygon which is shown on the left-hand side of the arch, represent the special equilibrium polygon for the second condition of loading.

Intensity of Pressures on the Voussoirs of the Arch. An inspection of the equilibrium polygon for the first condition of loading shows that it passes everywhere within the middle third. The maximum total pressure on a joint, of course, occurs at the abutment, where the pressure equals 24,750 pounds. Since the joint is here about 42 inches thick, and a section one foot wide has an area of 504 square inches, the pressure on the joint is at the rate of 49 pounds per square inch. At the keystone, the actual pressure is 19,750 pounds; and since the keystone has an area of 228 square inches, the pressure is at the rate of 87 pounds per square inch.

At the joint between forces Nos. 13 and 14, the line of force passes just inside the edge of the middle third. The ray from the pole o_1' to the joint between voussoirs Nos. 13 and 14 of the force diagram has a scaled length of 20,250 pounds. The joint has a total thickness of about 24 inches, and therefore an area of 288 square inches. This gives an average pressure of 70 pounds per square inch; but since the line of pressure passes near the edge of the middle third, we may double it, and say that the maximum pressure at the upper edge of the joint is 140 pounds per square inch. All of these pressures for the first condition of loading are so small a proportion of the crushing strength of any stone such as would be used for an arch, or even of the good quality of mortar which would, of course, be used in such a structure, that we may consider the arch, as designed, to be perfectly safe for the first condition of loading.

The special equilibrium polygon for the second condition of loading shows that the stability of the arch is far more questionable under this condition, since the special equilibrium polygon passes outside the middle third, especially on the left-hand haunch of the arch. The critical joint appears to be between voussoirs Nos. 4 and 5. The pressure at this joint, as determined by scaling the distance from the point o_2''' to the load line between forces Nos. 4 and 5, is approximately 24,500 pounds. The section of the equilibrium polygon parallel to this ray passes through the joint at a distance of a little

over three inches from the edge. On the basis of the distribution of pressure at a joint, the compression at this joint would be confined to a width of 9 inches from the upper edge, the pressure being zero at a distance of 9 inches from the edge. This gives an area of pressure of 108 square inches, and an average pressure of 227 pounds per square inch. At the upper edge of the joint, there would, therefore, be a pressure of double this, or 454 pounds per square inch. This pressure approaches the extreme limit of intensity of pressure which should be used in arch work; and even this should not be used unless the voussoirs were cut and dressed in a strictly first-class manner, and the joints were laid with a first-class quality of mortar.

The propriety of leaving the dimensions as first assumed for trial figures, depends, therefore, on the following considerations:

First. The loading assumed above for the uniformly distributed load is as great a loading as that produced by ordinary locomotives such as are used on the majority of railroads; while the locomotive requirements as assumed above are excessive, and are used on only a comparatively few railroads.

Second. If an equilibrium polygon had been started from a point nearer the intrados than the point m—using the same pole o_2'''—it would have passed a little below the point c, and likewise a little nearer the intrados than the point n. Although this would have brought the equilibrium polygon a little nearer to the intrados on the right-hand haunch of the arch, it would likewise have drawn it away from the extrados on the left-hand haunch. Although it is uncertain just which equilibrium polygon, among the infinite number which may mathematically be drawn, will actually represent the true equilibrium polygon, there is reason to believe that the true equilibrium polygon is the one of which the summation of the intensity of pressures at the various joints is a minimum; and it is evident from mere inspection, that an equilibrium polygon drawn a little nearer the center, as described above, will have a slightly less summation of intensity of pressure, although the intensity of pressure on the joints on the right-hand haunch will rapidly increase as the polygon approaches the intrados. It is therefore quite possible that the true equilibrium polygon would have a less intensity of pressure at the joint between voussoirs Nos. 4 and 5.

If it is still desired to increase the thickness of the arch so that

the line of pressure will pass further from the extrados, it may be done, approximately as indicated for a similar problem on page 411. Evidently, the keystone is sufficiently thick, and the voussoirs at the abutments also have ample thickness. The extrados must evidently be changed from an arc of a circle to some form of curve which shall pass through the same three points at the crown and the two abutments. This may be either an ellipse or a three-centered or five-centered curve. Although it will cause an extra loading on the haunches of the arch to increase the thickness of the arch on the haunches, and although this will cause the equilibrium polygon to rise somewhat, the rise of the equilibrium polygon will not be nearly so rapid as the increase in the thickness of the arch; and therefore the added thickness will add very nearly that same amount to the distance from the extrados to the equilibrium polygon. For example, by adding a little over three inches to the thickness of the arch at voussoirs Nos. 4 and 5, the distance from the equilibrium polygon to the extrados would be increased from three inches to six inches, and the maximum intensity of pressure on the joint would be approximately half of the previous figure. To be perfectly sure of the results, of course, the problem should be again worked out on the basis of the new dimensions for the arch.

The required radii for a multicentered arch which should have this required extrados, or the axes of an arc of an ellipse which should pass through the required points, are best determined by trial. The effect of the added thickness on the load line for the right-hand side of the arch will be to bring the load line nearer to the center of the voussoirs and, therefore, will actually improve the conditions on that side of the arch. Of course, when the concentrated load is over the right-hand side of the arch instead of the left, the form of the equilibrium polygon will be exactly reversed. It is quite probable that, for mere considerations of architectural effect, the revised extrados would be made the same kind of a curve as the intrados. This would practically be done by selecting a radius which would leave the same thickness at the crown, allow the required thickness on the haunches, and let the thickness come what it will at the abutments, even though it is needlessly thick.

Stability of Pier between Arches. The stability of the pier

the assumption of the concentrated locomotive loading on the left-hand end of the next arch which is at the right of the given arch, and the uniform loading over the right-hand end of the given arch. We therefore draw through the point m' a line of force parallel to mk, and also produce the line ln until it intersects the other line of force in the point s. A line from s parallel to R_2, therefore, gives the line of action of the resultant of the forces passing down the pier, for this system of loading. Since this system of loading will give the most unfavorable condition, or the condition which will give a resultant with the greatest variation from the perpendicular, we shall consider this as the criterion for the stability of the pier. The piers were drawn with a batter of 1 in 12, and it should be noted that the resultant R_2 is practically parallel to the batter line. If the slope of R_2 were greater than it is, the batter should then be increased. The value of R_2 is scaled from the force diagram as 55,650 pounds. The force R_2 is about 14 inches from the face of the pier, and this would indicate a maximum intensity of pressure of 221 pounds per square inch. This is a safe pressure for a good class of masonry work. The actual pressure on the top of the pier is somewhat in excess of this, on account of the weight of that portion of the arch between the virtual abutment at n and the top of the pier; and the total pressure at any lower horizontal section, of course, gradually increases; but, on the other hand, the weight of the pier combines with the resultant thrust of the two arches to form a resultant which is more nearly vertical than R_2, and the center of pressure, therefore, approaches more nearly to the axis of the pier. The effect of this is to reduce the intensity of pressure on the outer edge of the pier; and since the numerical result obtained above is a safe value, the actual maximum intensity of pressure is certainly safe.

ELASTIC ARCHES

Technical Meaning. All of the previous demonstrations in arches have been made on the basis that the arch is made up of voussoirs, which are acted on only by compressive forces. The demonstration would still remain the same, even if the arches were monolithic rather than composed of voussoirs; but in the case of an arch composed of voussoirs, it is essential that the line of pressure shall pass within the middle third of each joint, in order to avoid a

tendency for the joint to open. If the line of pressure passes very far outside of the middle third of the joint, the arch will certainly collapse. An elastic arch is one which is capable of withstanding tension, and this practically means that the line of pressure *may* pass outside of the middle third and even outside of the arch rib itself. In such a case, transverse stresses will be developed in the arch at such a section, and the stability of the arch will depend upon the ability of the arch rib to withstand the transverse stresses developed at that section. A voussoir arch is, of course, incapable of withstanding any such stresses. A monolithic arch of plain concrete could withstand a considerable variation of the line of pressure from the middle third of an arch rib; but since its tensile strength is comparatively low, this variation is very small compared with the variation that would be possible with a steel arch rib. A reinforced-concrete arch rib can be designed to stand a very considerable variation of the line of pressure from the center of the arch rib.

Advantages and Economy. The durability of concrete, and the perfect protection that it affords to the reinforcing steel which is buried in it, give a great advantage to these materials in the construction of arch ribs. Although the theoretical economy is not so great as might be expected, there are some very practical features which render the method economical. It is always found that, before any considerable transverse stresses can be developed in a reinforced-concrete arch bridge, the concrete will be compressed to the maximum safe limit while the unit stress in the steel is still comparatively low. Since a variation in the live load often changes the line of pressure from one side of the arch rib to the other, and thus changes the direction of the transverse bending, it becomes necessary to place steel near *both* faces of the arch rib, in order to withstand the tension which will be alternately on either side of the rib. Of course the steel which is—for the moment—on the compressive side of the rib will assist the concrete in withstanding compression, but this is not an economical use of the steel. There is, however, the practical economy and advantage, that the reinforcement of the

especially great in the construction of arches of long span, since in such a case the dead load is generally several times as great as the live load. Therefore, the maximum variation in the line of pressure produced by any possible change in loading is not very great; and any method which will permit the use of a higher unit pressure in the concrete is fully justified by the use of such an amount of steel as is required in this case.

Mathematical Principles. A complete and logical demonstration of the theory of elastic arches requires the use of Integral Calculus. The theory is too long and too complicated for insertion here. The student will be asked to accept as demonstrable, several equations derived by calculus methods. Numerical problems will be proposed and the application of the data of the problems to these equations will be fully illustrated. In the practical numerical application of Integral Calculus to these problems, it is necessary to make a summation of a series of quantities. Theoretically, the number of the quantities should be infinitely great and the quantities themselves infinitesimally small. It is found that sufficiently accurate solutions can be obtained with a comparatively small number of quantities, twenty, ten, or even five; but the greater the number the more accurate will be the results. The center line of the arch rib between the abutments must be divided into five (ten or twenty) divisions on each side of the center, but each division must be of such length that the length ds divided by its moment of inertia I is a constant. If the rib were of constant depth h throughout, then the moment of inertia would be constant and each length ds would be the same. But an arch rib is generally made deeper at the abutment than at the crown. If the arch consists of plain concrete or other homogeneous material, ds varies as h^3. Equation (36) shows that, when the concrete is reinforced, even though the sections are symmetrical, I varies as a function of h^3 and h^2, and in the more general cases the function is still more complicated. There is no direct and exact method of dividing the half-span length into a given number of variable lengths, each one of which shall be proportional to the mean value of the moment of inertia of that section. The problem can only be solved either by a series of trials and approximations or else by making, at the outset, an approximation which permits a direct solution and yet such that the effect of the

approximation on the final result is demonstrably small and perhaps within the uncertainties of the construction work. An illustration of this approximation will be given in the numerical problem which will now be worked out.

ILLUSTRATIVE PROBLEM

Segmental Arch of Sixty-Foot Span. Assume a segmental arch having a net span of 60 feet and a net rise to the intrados of 15 feet. The only practicable method of solution is to assume trial dimensions which previous experience has suggested to be approximately right, and then test the strength of such a design. To find the radius for the intrados which will fulfill these conditions, we may note from Fig. 238 that the angle $A'B'C'$ is measured by one-half of the arc $A'C'$, and therefore $A'B'C'$ is one-half a, but its natural tangent equals $15 \div 30$, or 0.5. The angle whose tangent is 0.5 is 26° 34′. Therefore a equals 53° 8′. To find the radius, we must divide the half-span (30) by the sine of 53° 8′, which makes the radius 37.50 feet.

Depth of Arch Ring. For the depth of the keystone, we can employ only empirical rules. The depth as computed from Equation (61) would call for a keystone depth of about 27 inches, which would be proper for an ordinary masonry arch; but considering the accumulated successful practice in reinforced-concrete arches, and the far greater reliability and higher permissible unit stresses which may be adopted, we may select about two-thirds of this—or, say, 18 inches—as the depth of the arch ring at the crown. We will also assume that the variable lengths ds have the ratios 1.00, 1.10, 1.21, 1.33, 1.46, 1.61, 1.77, 1.95, 2.14, 2.36, in which the several values are those of a geometrical progression of 1.1. We will now assume, as a first approximation, that the moments of inertia, instead of varying according to the comparatively simple relation shown in Equation (42) vary directly as h^3. Then h, the mean height for the abutment section, will equal $\sqrt[3]{2.36 \times 18^3} = 23.96$, which we may call 24 inches at the skewback line. The first value of I, 7,387, given in Table XXXI is computed from Equation (36), by calling $b=12$, $h=18$, $n=15$, and $A=1.00$, it being assumed that the reinforcement consists of 1-inch square bars, spaced 12 inches on centers, in both intrados and extrados. The other values of I are obtained by

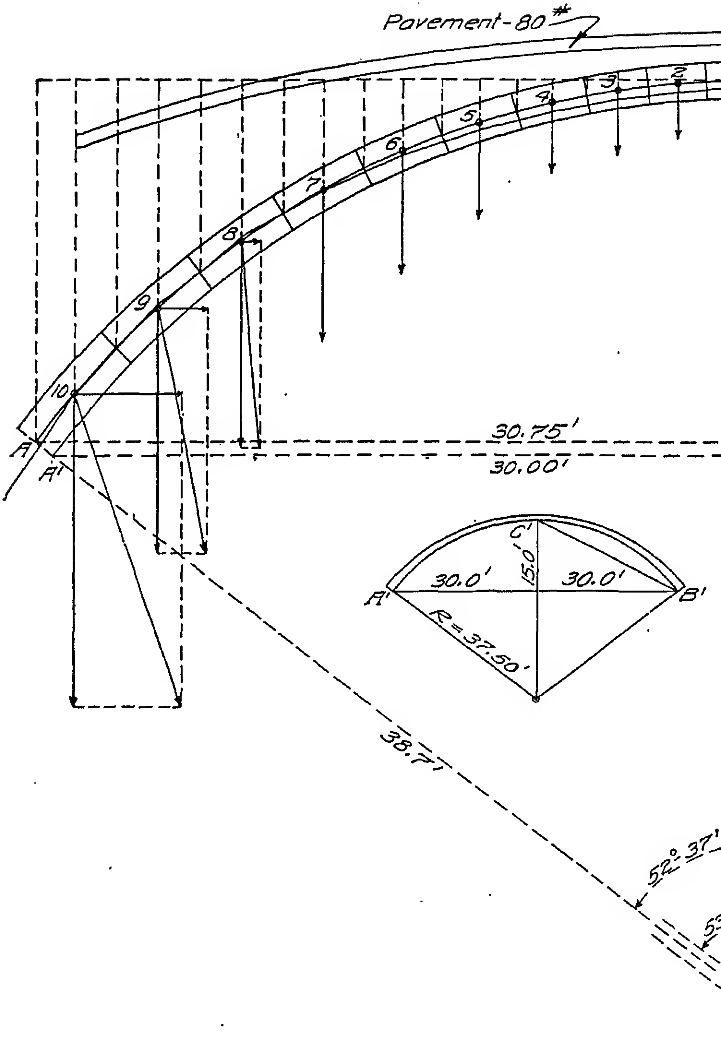


Fig. 238. Force Diagram for A

TABLE XXXI

Data for Segmental Arch, 60-Foot Span

(Illustrative Problem, page 440)

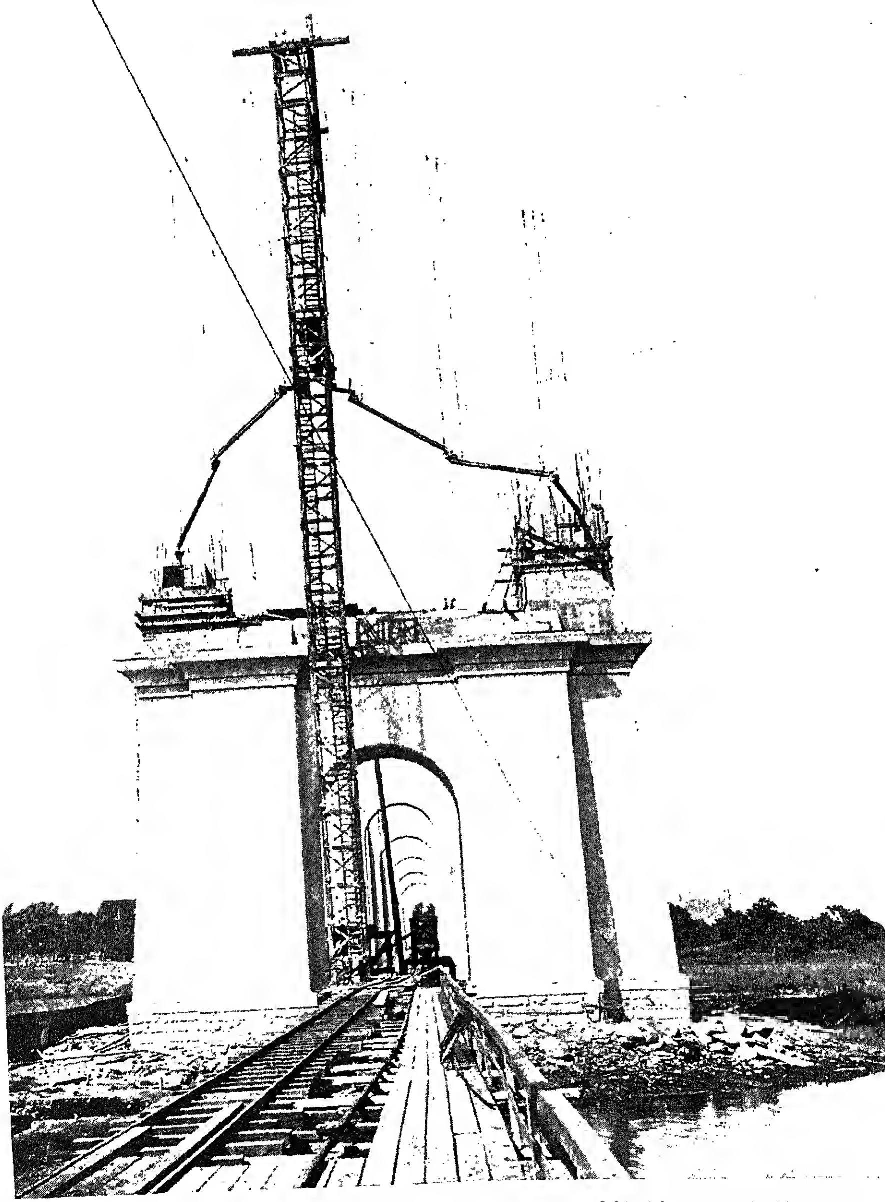
Point	ds Ratio	I	ds Feet	x	y
1	1.00	7,387	2.231	1.11	0.02
2	1.10	8,126	2.454	3.45	0.15
3	1.21	8,938	2.699	6.01	0.47
4	1.33	9,825	2.967	8.79	1.01
5	1.46	10,785	3.257	11.80	1.84
6	1.61	11,893	3.591	15.01	3.02
7	1.77	13,075	3.948	18.40	4.65
8	1.95	14,405	4.350	21.94	6.82
9	2.14	15,808	4.774	25.53	9.62
10	2.36	17,433	5.264	29.09	13.17
	15.93		35.535	30.75	15.20

multiplying 7,387 by the several values of the ds ratios. The approximation error, referred to above, will be greatest at the skew-back section. If we compute the I for the skewback section according to Equation (36), calling h equal to 23.96, we get I equals 16,511, about 5 per cent less than the tabular value, 17,433. $I_c = \frac{1}{12} b h^3 =$ 13,755, when $h = 23.96$ and by subtracting 13,755 from 17,433 we have 3,678, which we may place equal to $n I_s = 2n A \left(\frac{1}{2} h - d'\right)^2$, see Equation (35). Solving for d', we find that d' equals 0.91, or that by placing the bars 0.91 inch from the extrados and intrados, instead of 2.4 inches, the inertia requirement would be exactly satisfied. A very slight increase of thickness will not only cover the bars sufficiently but will also so increase the moment of inertia of the plain concrete that the inertia requirement will be exactly satisfied. Assume that h is increased *only one-half inch,* making it 24.46 inches; the moment of inertia becomes 17,506, which more than makes up the deficiency. Since the placing of the concrete might not be closer than this, our approximation is justified in this case and we may use the column of values for I and the corresponding ratio values of ds as they stand. It is thus practicable to assume, at least after one or two trials, a law of increase of ds which, by numerical test, will prove to be sufficiently close to the actual increase in the value of I and yet give a suitable increase in the depth of the arch ring—in this case an increase from 18 inches at the crown to 24 inches at the

Arch Rib Curve. The center line of the rib ACB must be 9 inches above the intrados at C and 12 inches at A and B, and is, approximately, an arc of a circle of somewhat larger radius than the radius of the intrados. The curve of the extrados has a still greater radius. The skewback lines AA' and BB' should be approximately perpendicular to both the intrados and extrados and for that reason we average it by making them perpendicular to the middle curve ACB. A and B are therefore located 12 inches from A' and B' and on lines through A' and B' from a center which must be first approximated and which is determined graphically by finding the center of a circle which will pass through C, A, and B, and whose radii will pass through AA' and BB'. Such a circle has a radius 38.7 and the scaled height of C above AB is 15.2 feet. This 15.2 is the versed sine of the angle AOC having a radius 38.7, from which AOC equals 52° 37′. Then the arc AC equals $52.61° \times 38.7 \times \text{arc } 1° = 35.535$ feet. Multiplying this half-span length by the ratio of each ds to 15.93 (the sum of the ds ratios) we have the values in the fourth column of Table XXXI. As a check, the sum of these computed values equals 35.535.

Strictly speaking, the intrados being an arc of a circle, the arch rib line, ACB, and the extrados line are probably not exact circles, but the approximation is here too small to be of importance. It would have been more simple to have assumed the span and rise of the arch rib line ACB, making it a true circle or some other definite curve, and laying off the half depth of the arch at each point to obtain the intrados and extrados curves. But this would not have permitted an exact preliminary requirement as to the precise form of the intrados, and the problem can only be solved this way by assuming an extra allowance which will prove to be sufficiently exact for the purpose. For example, in the above case, we might have chosen a span of 61.50 and a rise of 15.2 after making an approximate calculation that the result would give a *net* span of 60 and a net rise of 15. In fact, the arch rib curve is usually the one which is chosen. The other method was worked out to show how it might be done when for any reason the form of the intrados is strictly limited in its dimensions.

Loads on Arch. The dead load of a masonry arch is usually



VIEW OF APPROACH FOR NEW HELL GATE BRIDGE, SHOWING REINFORCED CONCRETE PIER CONSTRUCTION

load may be reduced by supporting the roadbed and the live load on columns or small piers extending from the deck down to the arch rib, which gives virtually the effect of concentrated loads at those points. The weight of the arch rib between these points may be considered as concentrated at these several points. The numerical problem considered indicates ten points on each side of the center. A solution can hardly be considered precise without having at least this number, but the numerical work involved is very great and very tedious. Therefore, in order to abbreviate somewhat, a solution will first be worked out in detail as if there were only three forces acting on the arch on each side of the center. Afterward, the solution for ten forces on each side of the center will be indicated as being worked out by the same method and the results will be given, but the details, which would require many pages, will be omitted. We will therefore assume that equal and symmetrical forces are applied at the points 2, 5, and 8 on each side of the center. Also, in order to cover another complication of the general problem, we will assume that while the forces at 2 are vertical, and equal 2,400, the forces at 5 and 8 are inclined and have horizontal and vertical components, those for 5 being 4,200 and 500 and those for 8 being 6,000 and 900. On pages 395, 403, and 420 are given in detail methods of computing, from the actual conditions of dead and live load, the amounts of the horizontal and vertical components of the forces at any point in the arch ring. The following numerical calculation, while much more condensed than would be proper for the complete investigation of an arch rib, even as small as that proposed above, will contain the complete method of work, and the more complete solution will only differ from it by having a far greater number of quantities and much more numerical work.

Forces at Any Section of Arch. The principles of graphical statics show that all the external forces lying in the plane of the arch rib and acting on any assumed section of the rib may be resolved into a force tangent to the rib curve, which we will call the *thrust*, T; also a force normal to the curve, which we will call the *shear*, S; and also a couple which produces a *moment*, M, about that point of the arch rib. The problem of the elastic arch is the determination of these forces at any section so that the sectional strength of the rib may be designed accordingly. By the application of Integral

demonstrable:

$$T_c = \frac{n\,\Sigma my - \Sigma m\,\Sigma y}{2\,[(\Sigma y)^2 - n\,\Sigma y^2]} \tag{62}$$

$$S_c = \frac{\Sigma\,(m_R - m_L)x}{2\,\Sigma x^2} \tag{63}$$

$$M_c = -\frac{\Sigma m + 2\,T_c \Sigma y}{2n} \tag{64}$$

in which T_c, S_c, and M_c are the thrust, shear, and moment, respectively, at the crown of the arch, m is the moment at any point, of all the loads between that point and the crown of the arch. The symbol Σ signifies the summation of a series of similar but variable quantities. Σy signifies the summation of all the values of y; $(\Sigma y)^2$ equals the square of the sum of all the y's; Σy^2 is the sum of the squares of each value of y. Note the distinction. Σmy is the sum of the products of each value of m times its corresponding value of y.

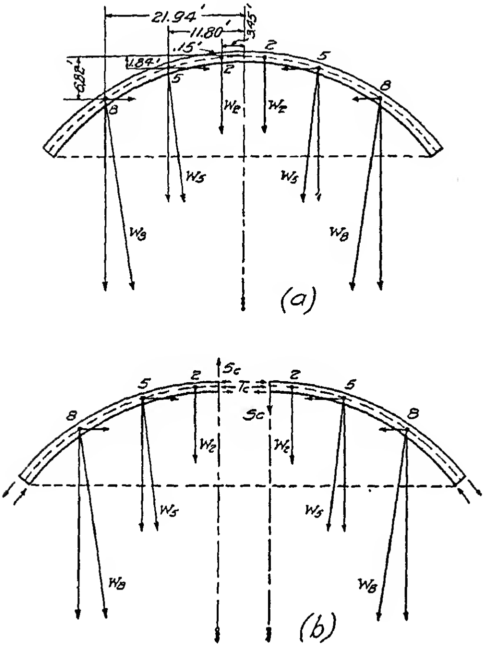


Fig. 239. Diagram of Forces in Segmental Arch

Assume a section made at the crown, and that equal and opposite forces (T_c, S_c, and M_c) are applied which will keep the two halves of the arch in equilibrium with the several external forces W and the abutment forces which are still unknown. When the moment forces at the crown are as shown in Fig. 239, there will be tension at the intrados and compression at the extrados. When the moment at the crown is in the direction indicated, it is called *positive*. Considering, now, the left

TABLE XXXII

Values of Quantities Used in Equations (62), (63), Etc.

Point	x	y	x^2	y^2	m_L	m_R	Σmy	$(m_R-m_L)x$
2	3.45	0.15	11.90	.02	0	0	0	0
5	11.80	1.84	139.24	3.39	2,004	2,004	7,375	0
8	21.94	6.82	481.36	46.51	89,454	89,454	1,220,153	0
		8.81	632.50	49.92	91,458	91,458	1,227,528	0
Abut.	30.75	15.20			224,072	224,072		

half of the arch, Fig. 239, as a cantilever, and taking moments about the point 8 of all the external forces, which in this case are W_2 and W_5, we will have

$$m \text{ (for point 8)} = 4{,}200\ (21.94-11.80)+2{,}400\ (21.94-3.45) + 500\ (6.82-1.84)$$

The moment is taken about each point in turn, as above. These moments are in each case in the contrary direction to the assumed (M_c) moment at the crown—whether for the left-hand or right-hand half of the arch—and are therefore considered as *negative*. Since each point of the left-hand side of the arch has the same y as the corresponding point on the right-hand side, $\Sigma my = \Sigma(m_L+m_R)y$. $\Sigma m = \Sigma(m_L+m_R)$. $\Sigma(m_R-m_L)x$ equals the sum of the products of the (m_R-m_L) for each pair of points, each multiplied by the common value of x for that pair. Having determined, after proper substitution of the numerical values in Equations (62), (63), and (64), the numerical values of T_c, S_c, and M_c at the crown, the value of M for *any* point, Q, may be determined from the equation

$$M = m + M_c + T_c y \pm S_c x \tag{65}$$

The negative sign is used with the last term when considering the right-hand half of the arch.

We will now apply these principles to the forces (named on page 447), which are applied at the points 2, 5, and 8. The several quantities needed for the solution of Equations (62), (63), and (64), and also some later equations, are given in Table XXXII.

The moment m for point 2 is zero (in *this* case) since there is no force between point 2 and the crown. For point 5, $m = 2{,}400\ (x_5-x_2) = 2{,}400\ (11.80-3.45) = 2{,}400\times 8.35 = 2{,}004$. For point 8, $m = 2{,}400(x_8-x_2) + 4{,}200(x_8-x_5) + 500(y_8-y_5) = 2{,}400\times 18.49 + 4{,}200 \times 10.14 + 500\times 4.98 = 89{,}454$. Although not used immediately, the

moments for the abutment points are calculated similarly and placed below. The student should verify this calculation for practice. These moments (m_L) are all negative, as stated above. Since, in this case, we have assumed the loads to be symmetrical, the moments m_R equal those of m_L, each to each. $\Sigma m y$ is found by adding the m_L and m_R for each point and multiplying by the corresponding value of y, thus obtaining the next column. The loads being symmetrical, $(m_R - m_L) = 0$ for each point, which gives a line of zeros for the last column, which means that the shear S_c in Equation (63) equals zero. This is only what might have been predicted—that there would be no shear at the crown when the loading is symmetrical. By substituting these values in Equation (62) and (64), we obtain numerical values for the crown thrust and moment as follows:

n equals 10; $\Sigma m.y$ equals $-1{,}227{,}528$; $\Sigma m \Sigma y$ equals $(-91{,}458 - 91{,}458) \times 8.81 = -182{,}916 \times 8.81$; $(\Sigma y)^2$ equals $8.81^2 = 77.616$; and $n \Sigma y^2$ equals $10 \times 49.92 = 499.2$. Then

$$T_c = \frac{10(-1{,}227{,}528) - (-182{,}916 \times 8.81)}{2(77.62 - 499.2)} = \frac{-12{,}275{,}280 + 1{,}611{,}490}{-843.16}$$

$$= \frac{10{,}633{,}790}{-843.16} = +12{,}647$$

As stated above, $S_c = 0$.

$$M_c = \frac{-182{,}916 + 2 \times 12{,}647 \times 8.81}{20} = -1{,}996 \text{ ft.-lb.}$$

This moment being negative, it indicates that there is compression at the intrados and tension at the extrados and that there is a tendency for the crown to rise, which is the invariable effect of heavy loads on the hamches and little or nothing on the crown.

It should be clearly understood that this very simple numerical solution only gives the stresses produced on the arch by the forces assumed. These forces do not allow for the weight of the arch. They are the stresses which would be produced if they were the only forces and the arch itself weighed nothing.

The moment at the abutment is found from Equation (65), in which m equals $-224{,}072$, and S_c equals 0. Then

$$M_A = -224{,}072 + (-1{,}996) + 12{,}646 \times 15.20 + 0 = -33{,}849 \text{ ft.-lb.}$$

The moment at point 5 is

Comparing the moments produced by these three forces at the crown, the abutment, and at point 5, they are negative at the crown and at the abutment but positive at point 5, showing that these forces create a tendency for the extrados to open (because of tension) at the crown and at the abutment, but there is tension in the intrados at point 5. This also means that there are two points on each side of the center, one between point 5 and the abutment and one between point 5 and the crown, where the moment changes sign and is zero.

Laying Off Load Line. We shall assume that the arch carries a filling of earth or cinders weighing 100 pounds per cubic foot, that the top of this filling is level, and that it has a thickness of one foot above the crown. Since concrete weighs about 150 pounds per cubic foot, we shall assume this weight of 150 pounds as the unit of measurement, and therefore reduce the ordinates of earthwork to the load line for the top of the earth. We shall assume, as an additional dead load, a pavement weighing 80 pounds per square foot, and shall therefore lay off an ordinate of $\frac{80}{150}$ of a foot above the ordinates for the earth-filling load. For this particular problem, we shall only investigate a live load of 200 pounds per square foot, extending over one-half of the span from the abutment to the center. From our previous work in arches, we know that such a loading will test the arch more severely than a similar unit live load extending over the entire arch; and therefore, if the arch proves safe for this eccentric load, we may certainly assume that it will be safe for a full load. The detail of the following calculation is worked out by precisely the same method as given in the previous article, but has been omitted here to save space. Although the calculations are long and tedious, the student will find that the surest method of grasping these principles is to work out and verify all the calculations of which the results only are given here. The horizontal projection of each ds, multiplied by the load-line ordinate in the line of each point, times 150, equals the vertical load on each joint. These loads are given in Table XXXIII. The horizontal component of each force is computed on the methods previously described for voussoir arches, the results being given in Table XXXIII.

Thrust, Shear, and Moment. Substituting in Equation (62),

TABLE XXXIII
Load Data for Segmental Arch Problem

Sec.	Hor. Comp. of ds	Left Half of Arch			x	y	x^2	y^2	Right Half of Arch		
		Reduced Load Line Ordin.	Load						Reduced Load Line Ordin.	Load	
			Vert. Comp.	Hor. Comp.						Vert. Comp.	Hor. Comp.
1	2.23	410	910	0	1.11	0.02	1.23	0.00	610	1,360	0
2	2.44	440	1,070	0	3.45	0.15	11.90	0.02	640	1,560	0
3	2.68	470	1,260	0	6.01	0.47	36.12	0.22	670	1,800	0
4	2.90	530	1,540	0	8.79	1.01	77.26	1.02	730	2,120	0
5	3.11	630	1,960	0	11.80	1.84	139.24	3.39	830	2,580	0
6	3.31	750	2,480	0	15.01	3.02	225.30	9.12	950	3,140	0
7	3.48	930	3,230	0	18.40	4.65	338.56	21.62	1,130	3,930	0
8	3.58	1,175	4,200	350	21.94	6.82	481.36	46.51	1,375	4,920	410
9	3.57	1,470	5,240	1,070	25.53	9.62	651.78	92.54	1,670	5,960	1,220
10	3.45	1,890	6,520	2,240	29.09	13.17	846.23	173.45	2,090	7,200	2,470
Σ	30.75				30.75	40.77	2,808.98	347.89			

$(-1{,}082{,}880)]\ 40.77 = (-1{,}862{,}315)\ 40.77 = -75{,}926{,}583$; $(\Sigma y)^2 = (40.77)^2 = 1{,}662.2$; and $\Sigma(y^2) = 347.89$. Then

$$T_c = \frac{-164{,}036{,}410 - (-75{,}926{,}583)}{2\,[1{,}662.2 - 10\times 347.89]} = \frac{-88{,}109{,}827}{-3{,}633.4} = +24{,}250 \text{ lb.}$$

The unit thrust at the crown under this condition of loading is only $24{,}250 \div 216 = 112$ lb. per sq. in. Then by Equation (**63**)

$$S_c = \frac{-7{,}082{,}939}{2\times 2{,}808.98} = \frac{-7{,}082{,}939}{5{,}618} = -1{,}261 \text{ lb.}$$

which is an insignificant shearing stress per square inch on 216 square inches. Then from Equation (**64**)

$$M_c = -\frac{(-1{,}862{,}315) + 2\times 24{,}250\times 40.77}{2\times 10} = -5{,}751 \text{ ft.-lb.}$$

The moment at *any* point is found from Equation (**65**). For example, for point 5 on the left-hand side we use the values of $m = -30{,}595$ and $S_c x = +(-1{,}261)\ 11.80 = -14{,}880$. Then

$$M_{L5} = -30{,}595 + (-5{,}751) + (24{,}250\times 1.84) + (-14{,}880)$$
$$= -6{,}606$$

$$M_{L7} = -83{,}490 + (-5{,}751) + (24{,}250\times 4.65) + (-1{,}261\times 18.40)$$
$$= +1{,}320$$

At the left-hand abutment, the moment is

$$M_{LA} = -325{,}850 + (-5{,}751) + (24{,}250\times 15.20) + (-1{,}261\times 30.75)$$

TABLE XXXIII—(Continued)

Load Data for Segmental Arch Problem

Sec.	m_L	m_R	$(m_L+m_R)y$	$(m_R-m_L)x$
1	0	0	0	0
2	−2,130	−3,180	−797	−3,622
3	−7,200	−10,650	−8,390	−20,735
4	−16,200	−23,770	−40,370	−66,540
5	−30,595	−44,370	−137,936	−162,545
6	−52,230	−74,600	−383,027	−335,774
7	−83,490	−117,170	−933,069	−619,712
8	−127,990	−175,350	−2,068,779	−1,039,078
9	−188,320	−253,560	−4,250,886	−1,665,577
10	−271,280	−380,230	−8,580,387	−3,169,356
Σ	−779,435	−1,082,880	−16,403,641	−7,082,939
Abut.	−325,850	−422,390		

At the right-hand abutment, the moment is

$$M_{RA} = -422{,}390 + (-5{,}751) + (24{,}250 \times 15.20) - (-1{,}261 \times 30.75)$$
$$= -20{,}765 \text{ ft.-lb.}$$

At the point 5, on the right-hand side

$$M_{R5} = -44{,}370 - 5{,}751 + (24{,}250 \times 1.84) - (-1{,}261 \times 11.80)$$
$$= +9{,}379 \text{ ft.-lb.}$$

At the point 7, on the right-hand side, the moment is

$$M_{R7} = -117{,}170 - 5{,}751 + (24{,}250 \times 4.65) - (-1{,}261 \times 18.40)$$
$$= +23{,}044 \text{ ft.-lb.}$$

The moment at the right-hand abutment (20,765 ft.-lb., or 249,180 in.-lb.) is evidently the maximum produced by this system of loading. Some of the work is simplified and is more easily understood by utilizing some of the principles of graphics. In Fig. 238 we lay off a load line, at some convenient scale, showing the loads given in Table XXXIII. The shear at the center, S_c, equal to $-1{,}261$, is laid off *downward* (being negative) from the dividing point O of the load line. Then the true pole distance (call it T_c) equal to 24,250 is laid off horizontally, giving the position of the pole, P, as shown in Fig. 238. $M_c \div T_c = (-5{,}751) \div (24{,}250) = -0.24$, the eccentric distance of the thrust at the crown. Laying off this distance *below* the crown center, and drawing a line parallel to PO. we have one section of the true equilibrium polygon. The remainder is drawn by the method previously explained for voussoir arches. Although the numerical computations are far more accurate than

those scaled from the drawing, it is found that one checks the other closely. The moment at any point equals the force, as shown by the proper ray of the force diagram, times the distance of the corresponding side of the equilibrium polygon from the rib center. The moment is negative at the crown and at both abutments, but positive on both haunches, only 1,320 at $L7$ but 23,044 at $R7$. The thrust at any point is given by that component of the corresponding ray of the force diagram which is parallel to the tangent at that point. Usually, the tangent component is so nearly parallel with the ray itself that they are substantially equal and the thrust is considered as measured by the ray itself. The thrust is, of course, maximum at the abutments, the ray parallel to the thrust at the right abutment scaling 38,900. The eccentricity equals the moment divided by the thrust and, for the right abutment, equals $20{,}765 \div 38{,}900 = .534$ ft. or 6.4 inches. Referring to page 241 and following, Part III on flexure and direct stress, the 6.4 equals e, while the h equals 24 inches. Then $e \div h = .266$; $p = 1 \div (24 \times 12) = .00347$. Then, according to the diagram, Fig. 112, k equals .785. Substituting in Equation (50) the values $M = 249{,}180$ in.-lb., $b = 12$, $h = 24$, $k = .785$, $n = 15$, $p = .00347$, and $a = .4\,h$ or 9.6, we may solve for c. Then

$$c = 249{,}180 \div \left[12 \times 576 \left(.1962 - .1027 + \frac{15 \times .00347 \times 2 \times 92.16}{.785 \times 576}\right)\right]$$

$$= 249{,}180 \div [6{,}912\ (.1962 - .1027 + .0212)]$$

$$= 314 \text{ lb. per sq. in.}$$

Then from Equation (47), since $k\,h = 18.84$,

$$s = 15 \times 314 \left(\frac{21.6 - 18.84}{18.84}\right)$$

$$= 4{,}710 \times .146$$

$$= 688 \text{ lb. per sq. in.}$$

Temperature Stresses. The provision which should be made for temperature stresses in a concrete arch is often a very serious matter, for the double reason that the stresses are sometimes very great, and that the whole subject is frequently neglected. It will be shown later that the stresses due to certain assumed changes of temperature may be greater than those due to loading. There is much uncertainty regarding the actual temperature which will be assumed

A steel bridge, with its high thermal conductivity, will readily absorb or discharge heat; and it is usually assumed that it will readily acquire the temperature of the surrounding air. On the other hand, concrete is relatively a nonconductor. No matter what changes of temperature may take place in the outer air, the interior of the concrete will change its temperature very slowly. One test bearing on this subject was conducted by burying some electrically recording thermometers in the interior of a large mass of concrete, and recording the temperatures as they varied for a period of ten months, which included a winter season. It was found that the total variation of temperature was but a few degrees.

It is probably safe to assume that even during the coldest of winter weather the temperature of the interior of a large mass of concrete will not fall below that of the mean temperature for the month. Since the Weather Bureau records for temperate climates show that the *mean* temperature for a month, even during the winter months, is but little if any below freezing, it may usually be assumed that for concrete a fall of 30 degrees below the temperature of construction—say 60°—will be a sufficient allowance. A rise of temperature to 90° F. is probably much greater than would ever be found in an arch of concrete. The earth and pavement covering protect the arch from the direct action of the sun. Even in the hottest day, the space under a masonry arch seems cool, and the real temperature of the masonry probably does not exceed 70°, even if the outer air registers 95°. Therefore, if we calculate the stress produced by a change of temperature of 30 degrees from the temperature of construction, we are probably exceeding the real stresses produced. Even if this extreme limit should be sometimes exceeded, it simply lowers, temporarily, the factor of safety by a small amount.

Let T_t be the thrust at the crown due to the assumed change in temperature; M_t, the moment at the crown due to the assumed change in temperature; E, the modulus of elasticity, which is here taken as that of the concrete, since the moment of inertia is that of the "transformed" section, or the equivalent concrete section; and I, the moment of inertia of the equivalent concrete section, which is variable but proportional to ds so that $ds \div I$ is constant. Since the foot unit has been used for all dimensions, we

must find a numerical value for $ds \div I$, by expressing I in biquadratic feet. Taking the first combination, since they are all equal, 7,387 biquadratic inches equals $(7{,}387 \div 12^4) = 7{,}387 \div 20{,}736 = 0.3562$ biquadratic feet. The value of ds corresponding to I equals 7,387 is 2.231 feet. Therefore, $ds \div I$ equals $2.231 \div 0.3562$ or 6.262. e is coefficient of expansion with temperature or .0000065 for both steel and concrete.

Analytical Mechanics and Calculus gives us the temperature equation

$$T_t = \frac{EI}{ds} \times \frac{Lne(t-t_o)}{2[n\Sigma y^2 - (\Sigma y)^2]} \tag{66}$$

The summations refer to one-half of the arch only.
Also

$$M_t = -\frac{T_t \Sigma y}{n} \tag{67}$$

The bending moment at *any* point due to temperature is

$$M = M_t + T_t y \tag{68}$$

$$= T_t y - \frac{T_t \Sigma y}{n}$$

$$= T_t\left(y - \frac{\Sigma y}{n}\right) \tag{69}$$

The equilibrium polygon for these temperature stresses is a horizontal line which is at a distance below the crown equal to $\frac{\Sigma y}{n}$. Where this line intersects the arch rib, there is no moment due to temperature, no matter how much change of temperature there may be. Above and below this line, the temperature moments have opposite signs.

Note that the denominator of the main term in Equation (66) is the same, but with opposite sign, as that in Equation (62). We can therefore use the same numerical value. Substituting, E equals 2,000,000 pounds per square inch, or 288,000,000 pounds per square foot; L equals 61.60; n equals 10; e equals .0000065; $(t-t_o)$ equals $+30°$ F.; $(ds \div I)$ equals 6.262; and $2[n\Sigma y^2 - (\Sigma y)^2]$ equals

Then

$$T_t = \frac{288{,}000{,}000 \times 61.60 \times 10 \times .0000065 \times 30}{6.262 \times (+3{,}633.4)} = 1{,}520 \text{ lb.}$$

$$M_t = -\frac{1{,}520 \times 40.77}{10} = -6{,}198 \text{ ft.-lb.}$$

It should be noted that this moment, produced by a *rise* of temperature of 30° above the temperature of construction is more than the moment produced at the center by the load over the half-span. Also that the algebraic sign is *negative*, showing that the moment produces compression at the intrados and that the arch tends to rise, due to this force. This is what we might expect when the temperature rises and expands the arch. Also note that for a fall of temperature of $(t-t_o)$ below the temperature of construction, $(t-t_o)$ would be negative, which would change the algebraic sign of the moment, and this is what we would expect.

Substituting in Equation (69), we have at *either* abutment

$$M = 1{,}520\ (15.20 - \frac{40.77}{10}) = +16{,}906 \text{ ft.-lb.}$$

Again it should be noted that this is nearly as much as the moment produced at the right-hand abutment by the load above considered. Also that for a rise of temperature, as in midsummer, these two moments at the right abutment are opposite in sign and relieve each other, the net moment being the algebraic sum or numerical difference. For a fall in temperature, the moments have the same sign and their numerical sum must be taken as the measure of stress.

The horizontal component of the thrust at each section is the same and equals the thrust at the crown—in this case, 1,520 pounds. At any other point it equals the thrust at the crown times the cosine of the angle of that point from the center. For the abutment, it equals $1{,}520 \times \cos 52° 37'$, or 923 pounds. For other points the thrust may be more easily obtained by a graphical method, i.e., draw a line representing the crown temperature thrust, at some scale. Let that line be the hypothenuse of a right-angled triangle; the other two lines being parallel to the tangent, and to the normal to the arch

ng the arch very slightly and this produces precisely the ect in altering the moment as an equivalent *fall* in tempera-ince the thrust is variable along the arch, we must consider age thrust. A thrust of c pounds per square foot on a span would produce a shortening of $c\,L \div E$, which would also be l by a fall of temperature of $-(t-t_o)$ degrees, whose effect $-c\,L\,(t-t_o)$. Therefore, we may substitute $-c\,L \div E$ for) in Equation (66) and obtain

$$T_s = -\frac{I}{ds} \times \frac{c\,L\,n}{2\,[n\,\Sigma y - (\Sigma y)^2]} \tag{70}$$

; this equation to our numerical problem, we will assume an thrust of 150 pounds per square inch or 21,600 pounds per ot equals c. The other quantities will be the same as those page 456 and following.

$$T_s = -\frac{1}{6.262} \times \frac{21{,}600 \times 61.60 \times 10}{3{,}633.4} = -585 \text{ lb.}$$

ess than 40 per cent of the stress due to a change of 30 n temperature. For a rise in temperature, these stresses eutralize each other; for a fall in temperature, they combine ce a greater stress.

bined Stresses for Above Loading. The worst combi-stresses on an arch occur in winter when the temperature normal. For a temperature 30 degrees below normal, and bove described loading on the half-span, we would have at abutment, M_{RA} equals $-20{,}765$; M (for temperature stress abutment) equals $-16{,}906$; M (for rib shortening at right) equals $-\frac{585}{1{,}520} \times 16{,}906 = -6{,}504$; which totals 44,175 ds, or 530,100 inch-pounds. The thrust due to live and l is 38,900; that due to a *fall* of temperature is a tension gative) and equals -923; that due to rib shortening is 923) $= -355$. The combined thrust is $38{,}900 - 923 - 355$

Dividing 530,100 by 37,622 we have 14.09 inches, which entricity for this combination of stresses. $e \div h = 14.09 \div 24$ Using the diagram, Fig. 112, for $e \div h = .587$ and $p =$

.00347, $k = .632$. Using Equation (50), making substitutions and solving for c, we have

$$c = 530{,}100 \div \left[12 \times 576\left(.158 - .067 + \frac{15 \times .00347 \times 2 \times 92.16}{.632 \times 576}\right)\right]$$
$$= 530{,}100 \div [6{,}912\ (.158 - .067 + .026)]$$
$$= 530{,}100 \div 808.7$$
$$= 655$$

Since $kh = .632 \times 24 = 15.2$, then according to Equation (47)

$$s = 15 \times 655\left(\frac{21.6 - 15.2}{15.2}\right) = 4{,}037 \text{ lb. per sq. in. tension}$$

$$s' = 15 \times 655\left(\frac{15.2 - 2.4}{15.2}\right) = 8{,}074 \text{ lb. per sq. in. compressive}$$

stress in the steel near the intrados.

It should be noted that the compressive stress in the concrete for this combination of loading and stresses is practically at the limit and that the steel serves a very useful purpose in assisting the compression. Also, that the steel on the tension side has a very low unit stress, but the percentage of reinforcement is not too high, since a lower percentage would increase very materially the unit compression in the concrete, which is now at its limit. The combined stresses at other points can be worked up similarly, with comparatively little additional computations, and this should be done for a complete investigation of the problem, but it is probably true that the above conditions represent the worst conditions and that the design, as approximated, is probably safe.

Although the investigation of another form of loading, such as a maximum load over the whole arch, will require another complete set of calculations and the drawing of another equilibrium polygon and force diagram, some of the work already done may be utilized so that the effort need not be altogether doubled.

Testing Arch for Other Loading. A live load of 200 pounds per square foot over the entire arch would unquestionably increase the thrust over the entire arch, especially at the abutments. The stress due to shortening will, of course, be increased in proportion to the increase in the thrust. The stress due to moment cannot be accurately predicted. Of course, such an examination and test for full loading should be made in the case of any arch to be constructed,

general, by identically the same method as was used above.

To test the arch for a concentrated loading, such as would be produced by the passage of a road roller, or, in the case of a railroad bridge, by an especially heavy locomotive, the test must be made by assuming the position of that concentrated load which will test the arch most severely. Ordinarily, this will be found when the concentrated load is at or near one of the quarter points of the arch. The only modification of this test over that given above in detail is in the drawing of the load line, but the general method is identical.

HINGED ARCH RIBS

General Principles. The construction of hinged arches of reinforced concrete is very rare, but is not unknown. We may consider that, structurally, they consist of curved ribs which have hinges at each abutment, and which may or may not have a hinge at the center of the arch. The advantage of the three-hinged arch lies in the fact that it is not subject to temperature stresses. The two-hinged arch is partially subject to temperature stresses, but not to the same extent as the fixed arch, since the arch rib is not held rigid at the abutments as in the case of the fixed arch. Practically the hinges are made by having at each hinge a pair of large cast-iron plates which are a little larger than the size of the rib, and which have at their centers a bearing for a pin of due proportionate size. The bearings are so made that one may turn, with respect to the other, about the axis of the pin through an angle of a very few degrees.

Arches have been made with a single hinge at the center. This eliminates all moment at the center. If one abutment settles with respect to the other, the center hinge might relieve the stress somewhat, especially if the settlement happened to be in the arc of a circle about the hinge. The two-hinged arch is less subject to the effect of settlement, and the effect would be zero, provided that the net distance between the hinges remained unchanged. The three-hinged arch is practically independent of both settlement stresses and temperature stresses, excepting those developed by the friction of the pins in their bearings. Theoretically, the three-hinged type has very great advantages, particularly if the foundations are not firm, and some settlement or yielding seems to be inevitable. But

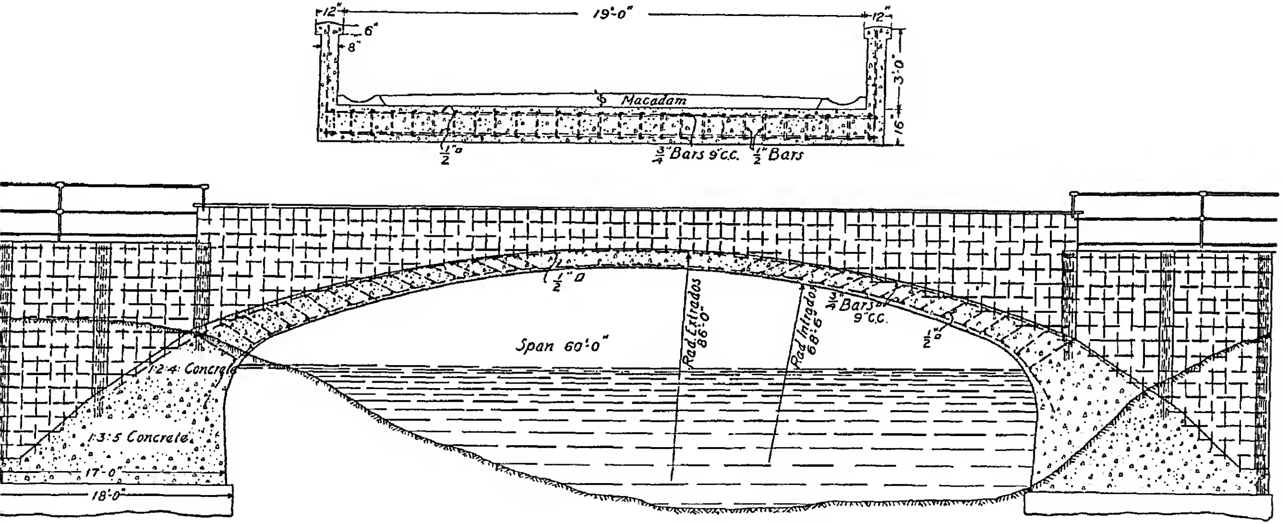


Fig. 240. Reinforced-Concrete Arch Rib of Berkley Bridge, Berks County, Pennsylvania

the hinges are, necessarily, very expensive features. The stresses produced in a fixed arch by arch settling may become indefinitely great and enough to produce complete failure. In spite of this fact and the immunity of the three-hinged type from such risk, comparatively few such arches have been built.

Description of Two Reinforced-Concrete Arches. *Berkley Bridge.* In Figs. 240 and 241 are shown the details and sections of two reinforced-concrete arches having fixed abutments. The first bridge, Fig. 240, has a nominal span of 60 feet between the two

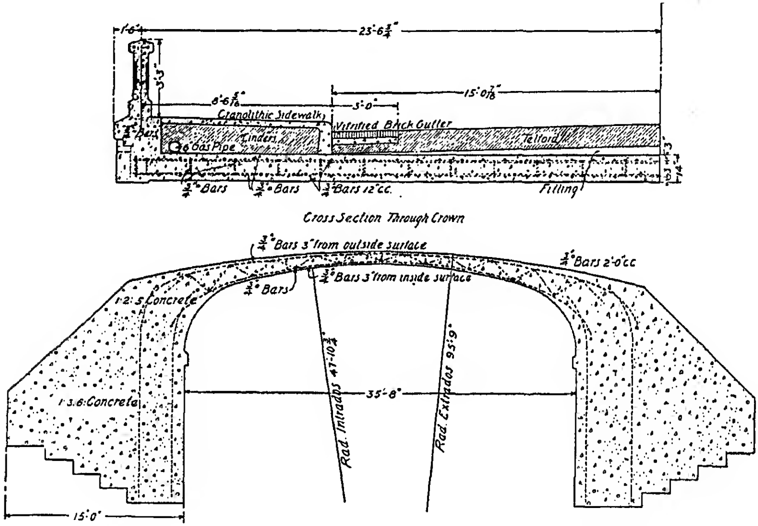
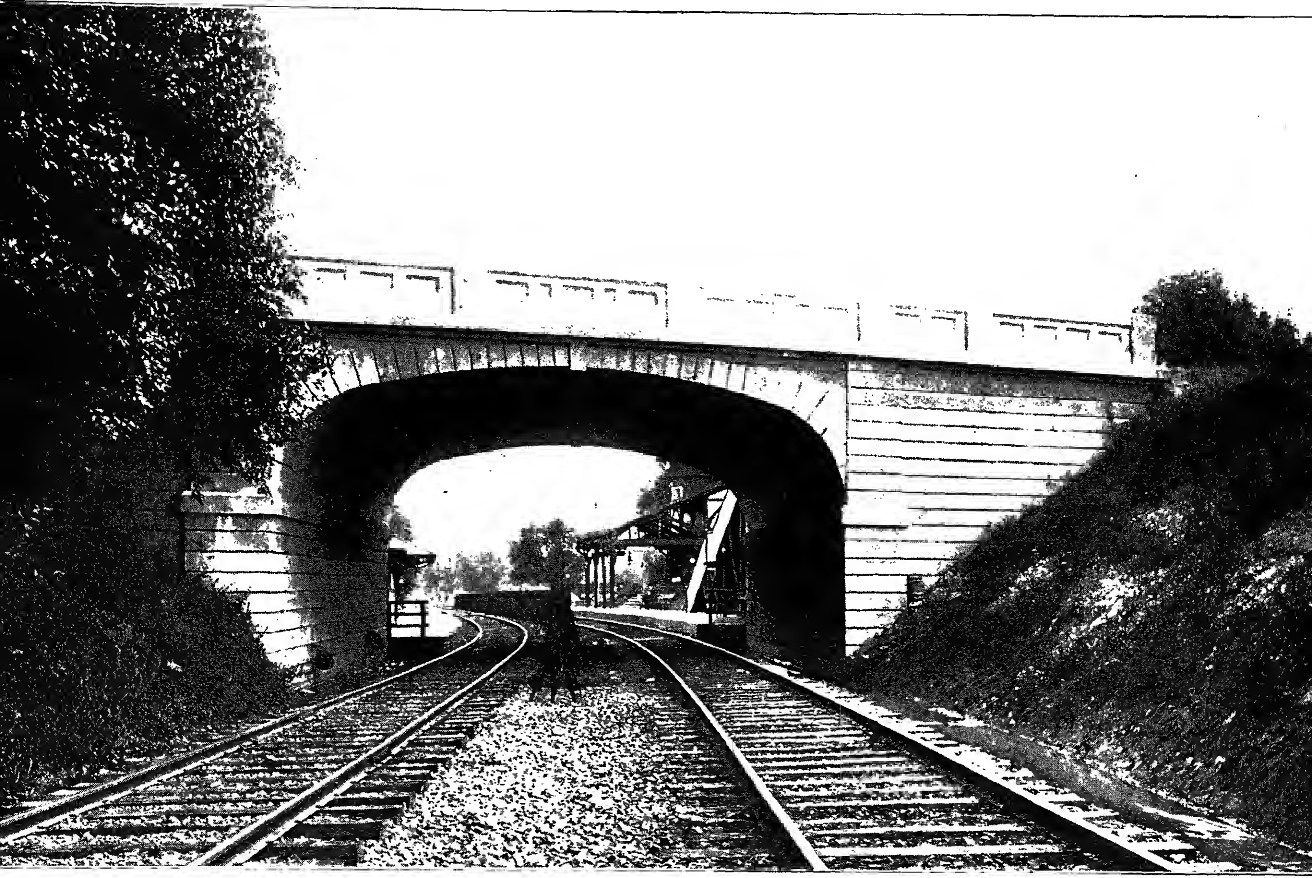


Fig. 241. Reinforced-Concrete Oblique Arch of Graver's Lane Bridge, Philadelphia, Pennsylvania

faces of the abutments. On account of the great thickening of the arch rib near the abutment, the virtual abutments are practically at points which are approximately 26 feet on each side of the center. The method of reinforcing the spandrel and parapet walls is clearly shown in the figure. The side view also gives an indication of some buttresses which were used on the inside of the retaining walls above the abutments in order to reinforce them against a tendency to burst outward.

Graver's Lane Bridge. Fig. 241 shows a bridge which is slightly oblique, and which spans a double-track railroad. The perpendicu-



CONCRETE ARCH AT GRAVER'S LANE, PHILADELPHIA, PENNSYLVANIA

the oblique face walls is 35 feet 8 inches. In this case, similarly, the arch is very rapidly thickened near the abutment, so that the virtual abutment on each side is at some little distance out from the vertical face of the abutment wall. In both of these cases, the arch rib was made of a better quality of concrete than the abutments.

The arch of Fig. 240 was designed for the loading of a country highway bridge; that of Fig. 241 was designed for the traffic of a city street, including that of heavy electric cars.

Stone Arch. In Fig. 242 is shown a stone arch on the New York, New Haven and Hartford Railroad at Pelhamville, New York. This arch was constructed over a highway, and the length of its axis is sufficient for four overhead tracks. The span is 40 feet, and the

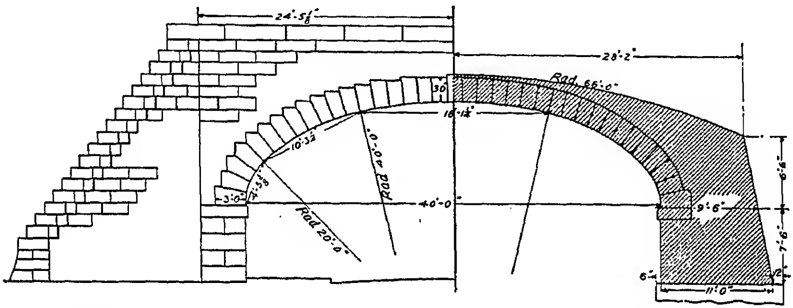


Fig. 242. Stone Arch on Line of New York, New Haven and Hartford Railroad

rise is 10 feet above the springing line, the latter being 7 feet 6 inches above the roadway. The length of the barrel of the arch is 76 feet.

The arch is a five-centered arch, the intrados corresponding closely to an ellipse, the greatest variation from a true ellipse being 1 inch. The theoretical line of pressure is well within the middle third, with the full dead load and partial live load, until the short radius is reached, where it passes to the outer edge of the ring stone, and thence down through the abutment. There is a joint at the points where the radii change, to simplify the construction.

The stone is a gneiss found near Yonkers, New York, except the keystone, which is Connecticut granite, and the coping, which is bluestone from Palatine Bridge, New York.

REVIEW QUESTIONS

REVIEW QUESTIONS

ON THE SUBJECT OF

MASONRY AND REINFORCED CONCRETE

PART I

---

1. Describe the tests that should be applied to determine the qualities of a building stone.

2. Describe the distinguishing characteristics of limestone, sandstone, and granite; and the uses for which these characteristics make them especially suitable.

3. Discuss the crushing strength of various kinds of brick.

4. Describe briefly the characteristics and method of manufacture of sand-lime brick.

5. Describe the essential features in the manufacture of concrete building blocks.

6. Describe the various changes that take place in transforming the original limestone into lime, and from that into the hardened mortar.

7. What is the essential characteristic of hydraulic lime?

8. What is the essential characteristic of slag cement, and for what kind of use is it especially suited?

9. What is the essential difference between natural cement and Portland cement?

10. If a certain brand of cement requires 30 per cent of water to produce a paste of standard consistency, how much water should be used in a 1 : 3 mortar?

11. What is "initial set"? How soon should it develop, and what is the standard test for the time?

12. How much tensile strength should be developed by briquettes of neat natural cement, and also by those of neat Portland cement, in 7 days? Also in 28 days?

13. What are the desirable characteristics of sand for use in mortar?

14. Why does sand with grains of variable size produce a

at are the characteristics of various kinds of broken
vel which have an influence on their value in concrete?
at practical method should be adopted to mix a large
ne mortar in the proper proportions?
ıme that the voids in the sand are measured to be
 40 per cent, and that the voids in the stone are
 45 per cent. Using barrels containing 3.8 cubic
, how much cement, sand, and stone will be required
yards of 1:3:6 concrete?
h cement at $1.25 per barrel, sand at $1.00 per cubic
ken stone at $1.40 per cubic yard, the cost including
e site of the work, what will be the cost on the mixing
bic yard of 1:3:6 concrete?
er what conditions is it proper to use dry concrete?
at is the danger in the excessive ramming of very

 is there any practical difficulty in bonding old and

at is the effect of the freezing of concrete before it is
n concrete be safely placed in freezing weather?
cribe in detail how you would make concrete water-
ng the proportions or by the use of cement grout.
cribe the method of waterproofing by the use of felt
r by the use of asphalt alone.
t form of bitumen should be used for waterproofing

uss the effectiveness of concrete in preserving
l from corrosion.
uss the protection afforded to imbedded steel by the
nst fire.
t precautions should be taken to insure that hand-
e is properly mixed?
uss the relative strength of machine-mixed and
ncrete.
t requirements should a high-carbon steel satisfy in
itable for reinforcing concrete?
t is the effect of using lime in cement mortar?
ribe the principles underlying the mixing of con-
btain the best possible product.

REVIEW QUESTIONS

ON THE SUBJECT OF

MASONRY AND REINFORCED CONCRETE

PART II

---

1. Define the different classes of masonry with respect to the dressing of the stones.

2. Give an outline of the method of dressing a stone which shall have a warped surface.

3. What is the purpose of bonding? Describe several ways in which it is accomplished.

4. A square pier in a building is to carry a load of 420,000 pounds; the pier is to be made of squared-stone masonry. What are the proper dimensions of the pier?

5. What are the elements affecting the cost of stone masonry?

6. Describe the various kinds of bonds used in brick masonry.

7. What tools are used, and how are they employed in the operation of quarrying and dressing stone for ashlar masonry?

8. Describe the various methods used in measuring brick-work.

9. A brick pier is 20 feet high; it is required to carry a load of 400,000 pounds, and is to be laid in a 1 to 2 natural cement mortar. Assume that the pier is to be square, what should be its cross-sectional dimensions?

10. Assuming that two-man stone is to be used in making rubble concrete, what will be the proper proportions of cement, sand, small broken stone, and rubble in such a concrete?

11. Describe the method of depositing concrete under water, using buckets.

12. What precautions must be taken when depositing concrete under water through a tube?

13. Describe the tests for determining the suitability of clay for use as clay puddle.

14. How would you test the bearing power of a soft soil?

15. Discuss the bearing power of various kinds of soil.

16. Describe some of the methods of improving a compressible soil.

17. Describe some of the methods of preparing the bed for foundations on various kinds of soil.

18. What is the purpose of a footing?

19. The wall of a building has a thickness of 2 feet; the total load on the wall has been computed as 16,000 pounds per running foot of the wall; the soil is estimated to carry safely a load of 3,000 pounds per square foot. What should be the thickness and width of limestone footings to support this wall on such a soil?

20. Classify the various kinds of piles, describing their uses.

21. Under what conditions do timber piles rapidly decay?

22. What are the most necessary specifications for timber piles?

23. A wall having a weight of 15,000 pounds per running foot is to be built on two lines of piles placed $2\frac{1}{2}$ feet apart transversely. It is found that piles driven 20 feet into such a soil have an average penetration for the last five blows of 1.5 inches, when a 2,500-pound hammer is dropped 24 feet. What is the bearing power of such piles, and how far apart must they be placed longitudinally in order to carry that wall?

24. Discuss the advantages and disadvantages of drop-hammer and steam-hammer pile drivers, and the use of the water jet.

25. What are the relative advantages and disadvantages of concrete piles compared with wood piles?

26. What is a grillage, and what is its purpose?

27. What combination of circumstances justifies the use of a cofferdam?

28. What is the essential disadvantage nvolved in the use of a crib as a foundation for a pier?

29. What general constructive principle is involved in the sinking of a hollow crib through a soft soil?

REVIEW QUESTIONS

ON THE SUBJECT OF

MASONRY AND REINFORCED CONCRETE

PART III

---

1. Why is there but little, if any, structural value to a beam made of plain concrete?

2. Develop a series of equations (similar to Equation **23**) on the basis of $1:2\frac{1}{2}:5$ concrete whose modulus of elasticity (E_c) is assumed at 2,650,000, and whose ultimate crushing strength (c') is assumed at 2,200 pounds.

3. Using a factor of 2 for dead load and a factor of 4 for live load, what is the maximum permissible live load which may be carried on a slab of $1:2\frac{1}{2}:5$ concrete with a total actual thickness of 6 inches and a span of 8 feet?

4. If a roof slab is to be made of 1:3:5 concrete and designed to carry a live load of 40 pounds per square foot on a span of 10 feet, what should be the thickness of the slab, and the spacing of $\frac{3}{8}$-inch square bars?

5. A beam having a span of 18 feet is required to carry a live load of 12,000 pounds uniformly distributed. Using 1:3:5 concrete and a factor of 4, what should be the dimensions of the beam whose depth is approximately twice its width?

6. What will be the intensity per square inch of the maximum vertical shear in the above beam?

7. What are the two general methods of providing for diagonal shear near the ends of the beam?

8. Make a drawing of the beam designed in Question 11, showing especially the reinforcement and the method of providing for the diagonal shear.

9. Make a design for a slab of 1:3:5 concrete, reinforced in both directions, which is laid on I-beams spaced 10 feet apart in each direction.

10. What is the general structural principle which makes T-beams more economical and efficient than plain rectangular beams having the same volume of concrete?

11. What assumption is made regarding the distribution of compressive stress in a T-beam?

12. How is the width of the flange of a T-beam usually determined?

13. What principles govern the determination of the proper width of the rib of a T-beam?

14. Make complete drawings of the reinforcement of the floor slabs and beams (Question 20), making due provision for shear, and making all necessary checks on the design as called for by the theory?

15. What will be the bursting stress per inch of height at the bottom of a concrete tank having an inside diameter of 10 feet, designed to hold water with a depth of 40 feet? What size and spacing of bars will furnish such a reinforcement?

16. With a nominal wind pressure of 50 pounds per square foot, on a flat surface, what will be the intensity of the compression on the leeward side of the tank, allowing also for the weight of the concrete, and assuming a thickness of 12 inches?

17. On the basis of the approximate theory given in the text, what would be the required steel vertical reinforcement for the above described tank?

18. Design a retaining wall to hold up an embankment 30 feet high, making a cross-sectional drawing and plan drawing similar to Fig. 113, assuming that the buttresses are to be 12 feet apart.

19. Compute the required detail dimensions and the reinforcement for the box culvert illustrated in Fig. 119, on the basis that the culvert is to be 10 feet wide, 12 feet high, supporting an embankment 15 feet deep, and also a railroad loading of 1,500 pounds per square foot.

20. A column is to be supported on a soil on which the safe load is estimated at 6,000 pounds per square foot; the column carries a total load of 210,000 pounds; the column is 22 inches square; what should be the dimensions of the footing, and how should it be reinforced?

REVIEW QUESTIONS

ON THE SUBJECT OF

MASONRY AND REINFORCED CONCRETE

PART IV

---

1. What are the difficulties encountered in obtaining a satisfactory outer surface of concrete?
2. Describe two successful methods of obtaining a good outer surface.
3. When and how can acid be properly used in treating a concrete surface?
4. What pigments should (and should not) be used for coloring concrete?
5. Describe the various methods of finishing concrete floors.
6. How may efflorescence be removed from masonry surfaces?
7. What are the practical difficulties and disadvantages of measuring the materials of concrete in the operation of automatic measuring machines?
8. Make a sketch and plan for the concrete plant for a 6-story building, 40 feet by 100 feet; or, describe, with comments and sketch, the plant of some similar building actually being erected.
9. What precautions are taken to prevent the lumber in the forms from swelling or buckling?
10. Describe various devices for holding column forms together.
11. How are I-beams utilized to support the forms for concrete slabs laid on them?
12. Make a sketch design for the forms for a vertical wall ten feet high, six inches thick, and twenty feet long.
13. Describe the methods of lowering the centering under arches.
14. What should be the dimensions of a column of hemlock 12 feet high, to support safely a load of 15,000 pounds?
15. What are the several methods of bonding old and new

REVIEW QUESTIONS

ON THE SUBJECT OF

MASONRY AND REINFORCED CONCRETE

PART V

---

1. Draw the intrados for a segmental arch with a span of 40 feet and a rise of 10 feet. Compute the proper depth of keystone; make the thickness at the abutment $\frac{1}{4}$ greater, and draw the extrados. Use scale of $\frac{1}{2}$ inch as equal to 1 foot.

2. On the basis of Question 8, draw the load line, allowing for a level cinder fill, a 7-inch pavement, and a live load of 200 pounds per square foot.

3. Assuming 15 voussoirs in the above arch, compute the vertical loads on each voussoir, and draw a *half* load line for *full loading* over the whole arch. Use scale of 3,000 pounds per inch for load line.

4. Determine the special equilibrium polygon for the above loading, and the maximum unit-intensity of pressure at any joint.

5. Determine the load line for a concentrated load of 20,000 pounds on an area of 25 square feet at the quarter-point of the arch, and a load of 200 pounds per square foot over the remainder of the *half-span*.

6. Draw the special equilibrium polygon for the loading of Question 12, and determine the maximum unit-intensity of pressure at any joint.

7. Design an abutment for the above arch which shall be stable under either of the above conditions of loading.

8. Draw the load line for the above arch on the basis of the loading of Question 12, but on the assumption that the pressures on the arch are perpendicular to the extrados.

9. Redraw the extrados and intrados of Fig. 230 on the scale of $\frac{1}{4}$ inch equals 1 foot; and then, by scaling the various thicknesses at every two-foot section, for 26 feet on each side of the center, compute the moment of inertia for each section.

10. On the basis that Fig. 230 is virtually a segmental arch with abutments 26 feet each side of the center, determine the posi-

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